Toll Bridge Program Oversight Committee
Department of Transportation
Office of the Director
1120 N Street
P.O. Box 942873
Sacramento, CA 94273-0001

August 13, 2013

To: Bay Area State Legislative Delegation
Bay Area Congressional Delegation
Bay Area Toll Authority
California Transportation Commission
California State Transportation Agency

Fr: Steve Heminger, Chair, Toll Bridge Program Oversight Committee (TBPOC)

Re: Update on San Francisco-Oakland Bay Bridge East Span Seismic Safety Replacement Project

On July 10, 2013 at a meeting of the Bay Area Toll Authority (BATA), members of the Toll Bridge Program Oversight Committee (TBPOC) briefed BATA commissioners on findings contained in the recently released report on the high-strength steel bolts used on the new Bay Bridge East Span. A key focus was the ongoing effort to retrofit Pier E2 where the bolt failure occurred. The fabrication and installation of a reinforcing steel saddle is forecast for completion by December of this year.

The Toll Bridge Seismic Safety Peer Review Panel (TBSSPRP) provided a graphic showing the superior strength of the new East Span design compared to the old East Span, and presented a proposal for an interim fix at Pier E2. The TBPOC asked the Federal Highway Administration (FHWA) and two preeminent bridge engineers from the firms of Buckland & Taylor, Ltd., and Modeski and Masters to review this recommendation. All reviews have reached the same and unequivocal conclusion that the interim retrofit will adequately protect and allow for the opening of the new East Span while the permanent retrofit is under construction.

The TBPOC is submitting the following information to policy leaders for your information. Attached are the following:

- Seismic Evaluation of SAS at E2 Pier Prior to Completion by TY Lin International/Moffat Nichol Engineers, Joint Venture, Dated July 15, 2013
- Letter from TBSSPRP
- Letter and Report from FHWA
- Report from Buckland & Taylor, Ltd
- Report from Modjeski and Masters

The TBPOC will hold a public meeting at 10:00 AM on Thursday, August 15, 2013 at the Joseph P. Bort MetroCenter at 101 8th Street in Oakland, CA to discuss these reports and, in light of their positive conclusions, to take action on an opening date for the new East Span.

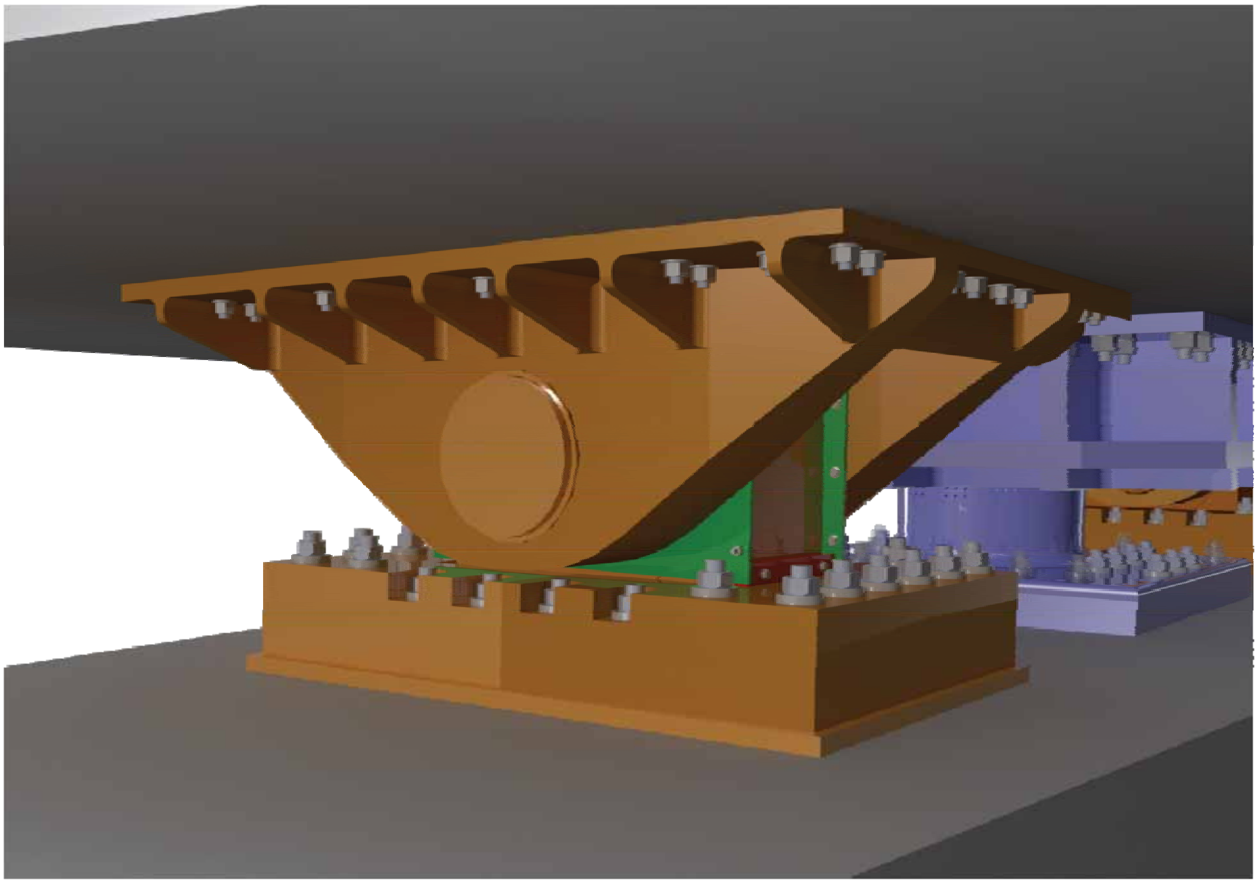
If you have any questions or concerns, please do not hesitate to contact me at (510) 817-5810.

Sincerely,

Steve Heminger
TBPOC Chair

San Francisco-Oakland Bay Bridge

Self-Anchored Suspension Span (SFOBB-SAS)



SEISMIC EVALUATION OF SAS AT E2 PIER PRIOR TO COMPLETION OF SHEAR KEYS S1 & S2

July 15, 2013

T·Y·LIN INTERNATIONAL
MOFFATT & NICHOL ENGINEERS
a joint venture

TABLE OF CONTENTS:

STATEMENT OF PURPOSE:

This report provides a summary of the technical information for the seismic evaluations of the San Francisco-Oakland Bay Bridge (SFOBB) East Spans Self-Anchored Suspension (SAS) at E2 Pier prior to completion of shear keys S1 and S2.

This report documents information on demand and capacity of relevant stages of construction and service. Additionally, visual images are included to support the understanding of various structural elements and staging. Supporting finite element analysis (FEM) is also provided.

This report evaluates temporary bearing modifications by adding simple shims to the Pier E2 Bearings (B1, B2, B3 and B4) to engage the bearing's reserve capacities for an interim condition.

EXECUTIVE SUMMARY:

As requested by Caltrans and as presented and discussed during the Seismic Safety Peer Review Panel (SSPRP) meeting with Caltrans and the peer review panel on July 3 2013, the Design Joint Venture of T.Y. Lin International / Moffatt & Nichol Engineers have performed an evaluation of the seismic capacity of the shear keys and bearings at Pier E2 of the Self-Anchored Suspension (SAS) Bridge. To this end various alternative load paths were evaluated and compared against the Seismic Demands for the Design Level Earthquake per the Project Specific Design Criteria. These Seismic Demands correspond to the envelope of the maximum time-history analysis response from six different 1500-year ground motions (SEE - Safety Evaluation Earthquake). At the top of Pier E2, these SEE demands total 50MN in the longitudinal direction of the bridge and 120MN in the transverse direction of the bridge.

The design lateral capacity of the shear keys and bearings at Pier E2 can be summarized as follows:

	Longitudinal Direction	Transverse Direction
- Shear Keys S1 & S2:	42 MN	42 MN
- Shear Keys S3 & S4:	42 MN (20mm Gap)	42 MN
- Bearings B1, B2, B3 & B4:	15 MN (20 mm Gap)	30 MN (20 mm Gap)

The design plans account for two alternative load paths:

A) Load Path A (shear keys are engaged) – This load path maintains the 20 mm gaps in S3 & S4 and the Bearings B1, B2, B3 & B4, thereby engaging only shear keys S1 & S2 in both directions and S3 & S4 in the transverse direction only. This provides a total capacity of 84 MN and 168 MN in the longitudinal and transverse directions, respectively.

B) Load Path B (all shear keys discounted) – This load path engages the Bearings B1, B2, B3 & B4 in both directions upon closing of the 20 mm gap due to seismic movement. This provides a total capacity of 60 MN and 120 MN in the longitudinal and transverse directions, respectively.

Assuming that the New Design of the Shear Keys S1 & S2 is not completed and by implementing interim shimming of the Bearings B1, B2, B3 & B4 to close the 20 mm gaps, a third alternative load path to resist the design lateral SEE demands can be developed: (reference Plan Sheet 883S1/1204 "Pier E2 Details No. 1A)

C) Load Path C (shear keys S1 & S2 discounted) – This load path engages the Bearings B1, B2, B3 & B4 by interim shimming of the 20 mm gaps in both directions, in addition to S3 & S4 being engaged in the transverse direction only. This provides a total capacity of 60 MN and 204 MN in the longitudinal and transverse directions, respectively.

The table in the Evaluation of Alternative Load Path at Pier E2 section provides a summary of the Seismic Lateral Capacity at Pier E2 for Load Path A, B & C, the SEE demands, and the associated Factors of Safety.

Enclosed please find a rendering depicting the installation sequence of the shims as well as a Finite Element Analysis (FEM) of the bearings.

BRIAN MARONEY'S (CALTRANS) MEMO:

(FROM EMAIL DATED JUNE 29, 2013 TO PMT / TBPOC / SSPRP)

This memo is to briefly summarize the safety of the Self-Anchored-Suspension bridge segment with respect to the expected performance of the San Francisco-Oakland Bay Bridge during a design level earthquake assuming the S1 and S2 shear key work currently underway is not fully completed by the time of seismic safety opening. In simplified terms, the bridge system between the orthotropic box girder superstructure and the concrete Pier E2 bentcap has enough strength capacity to carry 1500 year return period design level earthquake motion generated shear forces, overwhelmingly driving a shift of public traffic to the replacement bridge from the old bridge based on a desire for public safety.

The bridge capacity to carry the demand loads at Pier E2 is overdesigned to 140% of the worst of six different 1500 year return period earthquake time-history generated loads. The design criteria of the East Spans of the Bay Bridge is based upon 1500 year return period motions, which excides the national standards of 1000 year return period motions. This can be read as there is a 40% extra capacity in the "as-designed" system at Pier E2 above the lifeline criteria that is above the national standard. In simple terms, the system at Pier E2 was not designed to the bare minimum and there was a significant reserve capacity incorporated into the design that we should recognize at this time as leaders consider opening day alternatives. This extra design reserve is important to recognize when accounting for the fact that in construction there has developed a temporary reduction in capacity due to the Pier E2 threaded rod problem. The temporary reduction in strength capacity of the Pier E2 system due to the 2008 rod fractures is less than the overdesign. Therefore, leadership can advance increase public safety by opening the bridge as soon as feasible.

From bridge computer demand analysis models, earthquake lateral demands at the top of Pier E2 can be very simply summarized as 120 MN of force transversely and 50 MN of force longitudinally. If it is conservatively assumed that the S1 and S2 shear keys are completely ineffective, the S3 and S4 shear keys are only effective in the transverse direction and the B1, B2, B3 and B4 bearings are temporarily shimmed to engage them at zero relative displacement, lateral capacity to carry the 120 MN lateral demand is estimated at [2 * (42) + 4 * (30)] = 204 MN. Clearly, 204 MN is greater than 120 MN. Similarly, in the longitudinal direction the four shimmed bearings provide a capacity of [4 * (15)] = 60 MN and 60 MN is greater than 50 MN. These simple calculations demonstrate the new bridge provides well-above standard seismic safety even if the S1 and S2 shear key work is not complete.

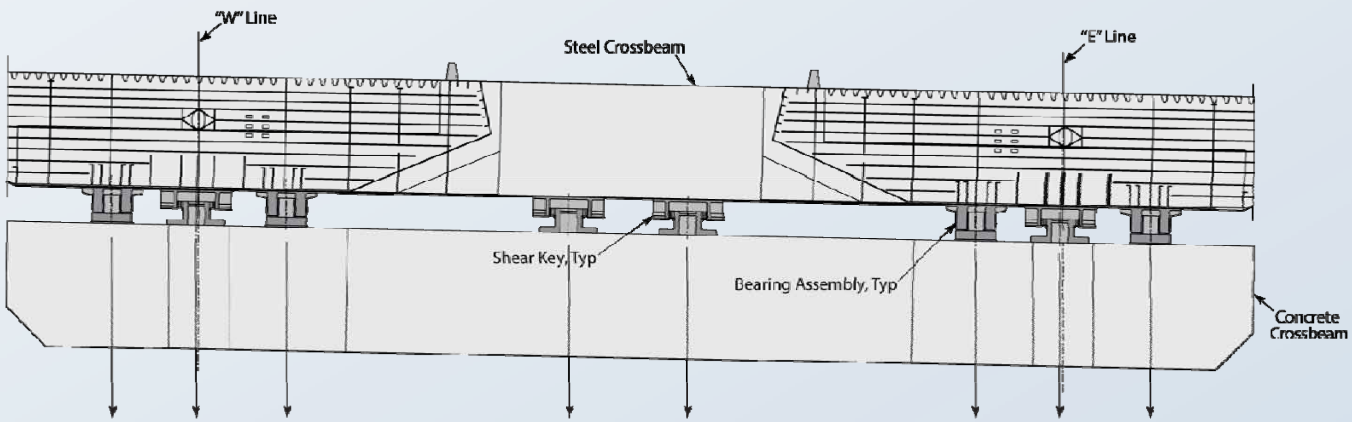
The existing bridge was not designed for the most basic "no-collapse" seismic safety criteria that is typically employed in modern bridge design. The old bridge is at risk in large Bay Area earthquakes as was demonstrated during the 1989 Loma Prieta Earthquake. The modest interim retrofit was developed to address the most fundamental seismic risks up to a limit of 25 million dollars. It was a good investment but was never intended to address long-term seismic risks associated with even a standard of 1000 year return period "no-collapse" criteria.

This summarizing discussion demonstrates that the San Francisco – Oakland Bay Bridge East Spans Replacement Structure offers significantly superior seismic safety to the public compared to the old bridge. From a technical perspective, it can be relatively easily concluded that the public should be moved onto the new structure at the first practical opportunity even if the S1 and S2 shear key work is not complete. It should be clear that the S1-S2 work is valuable as it provides the level of extra safety, reliability and toughness that was envisioned in the original design by bridge earthquake specialists and should be completed on an expedited schedule.

Evaluation of Alternative Load Path at Pier E2

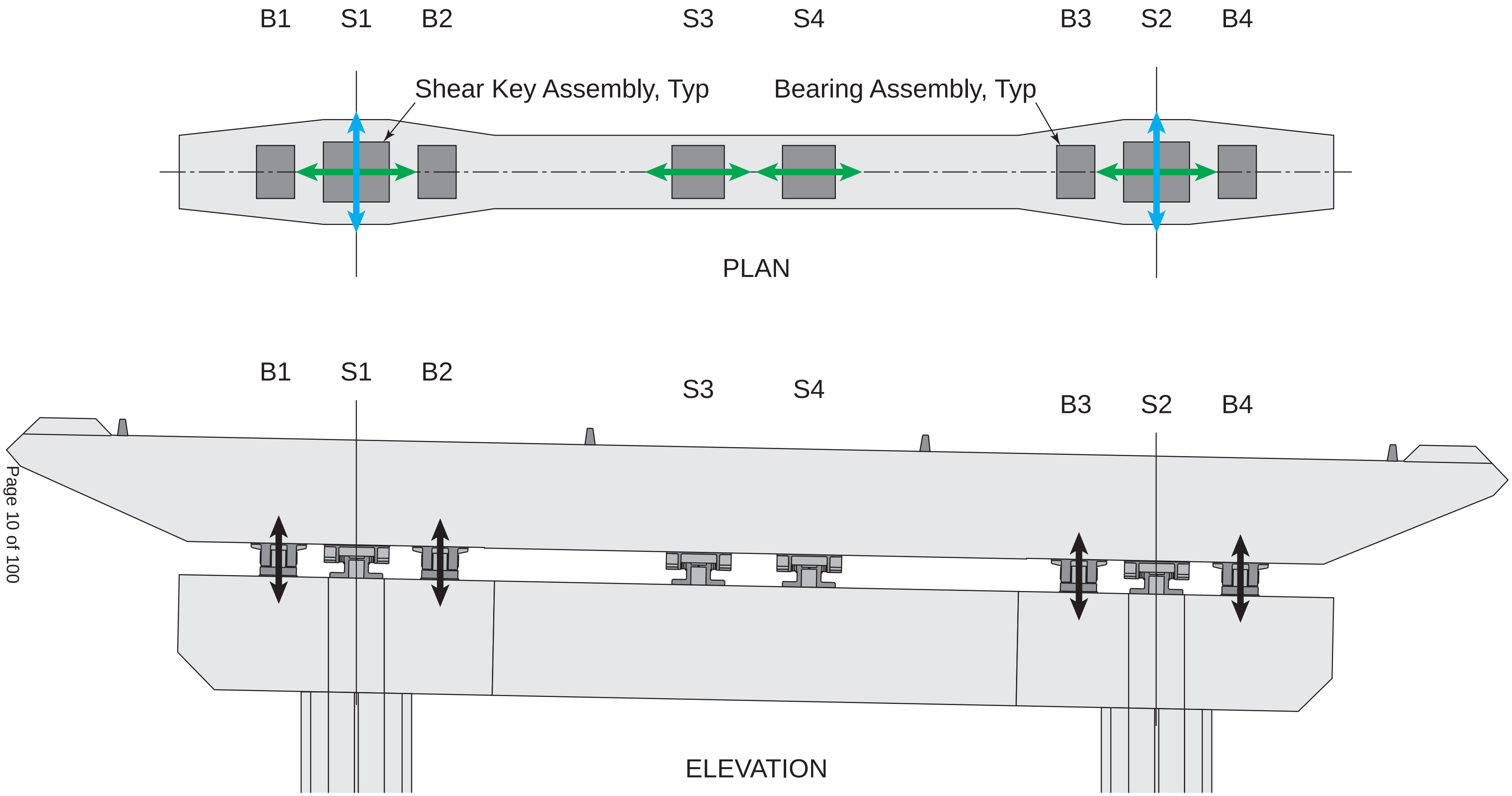
EVALUATION OF ALTERNATIVE LOAD PATHS AT PIER E2 FOR SEISMIC RESISTANCE

THE SAN FRANCISCO-OAKLAND BAY BRIDGE SEISMIC SAFETY PROJECTS



		Direction	B1	S1	B2	S3	S4	B3	S2	B4	TOTAL (MN)	SAFETY FACTOR
DEMAND	SEISMIC DEMAND FOR SEISMIC SAFETY EVALUATION (SEE) EARHQUAKE	Long									50	
		Trans									120	
CAPACITY	LOAD PATH A: (ALL SHEAR KEYS ENGAGED)	Long	0[a]	42	0[a]	0[c]	0[c]	0[a]	42	0[a]	84	168%
		Trans	0[b]	42	0[b]	42	42	0[b]	42	0[b]	168	140%
	LOAD PATH B: (ALL SHEAR KEYS DISCOUNTED) (BEARINGS ENGANGED)	Long	15	0	15	0	0	15	0	15	60	120%
		Trans	30	0	30	0	0	30	0	30	120	100%
	LOAD PATH C: (INTERIM SHIM OF BEARINGS) (S1 & S2 SHEAR KEYS DISCOUNTED)	Long	15	0	15	0[c]	0[c]	15	0	15	60	120%
		Trans	30	0	30	42	42	30	0	30	204	170%

a. 30 mm gap in the longitudinal direction. Bearing (B1, B2, B3, and B4) engage after 30 mm gap is closed by displacement.
b. 20 mm gap in the transverse direction. Bearing (B1, B2, B3, and B4) engage after 20 mm gap is closed by displacement.
c. 43 mm gap filled with neoprene open cell. Shear Keys (S3 and S4) engage in the longitudinal direction after 43 mm gap is closed by displacement.



LOAD PATH A — ALL SHEAR KEYS ENGAGED

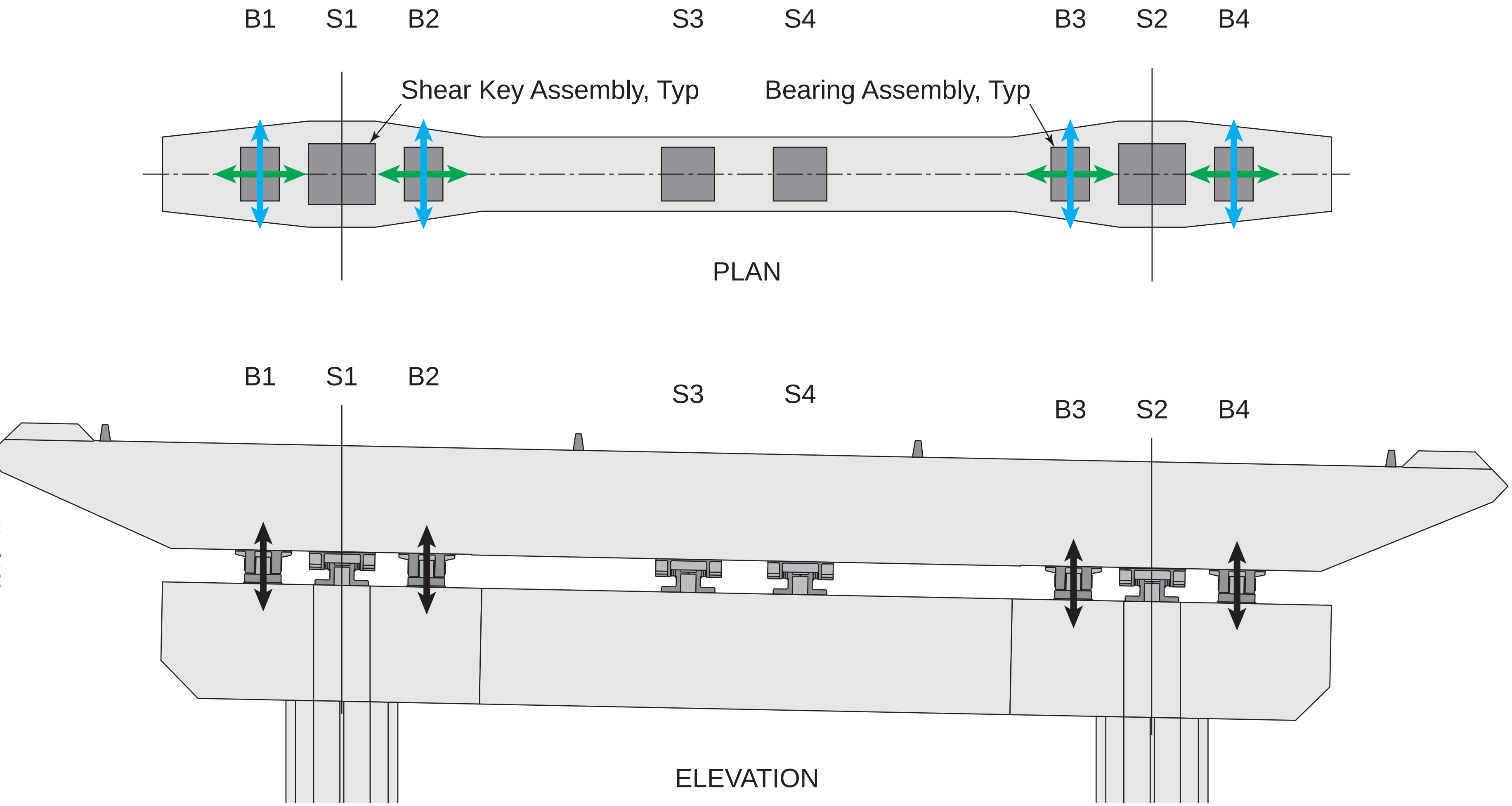
(Force Resistance Of Shear Keys And Bearings)

Direction of applied force that element can resist:

Uplift

Transverse

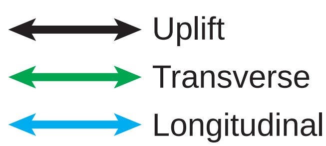
Longitudinal

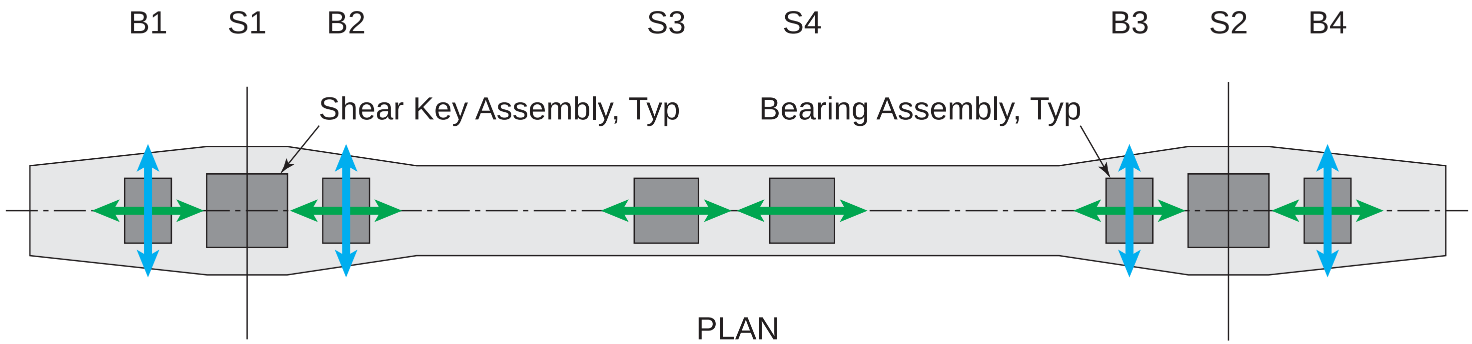


LOAD PATH B — ALL SHEAR KEYS DISCOUNTED / ALL BEARINGS ENGAGED

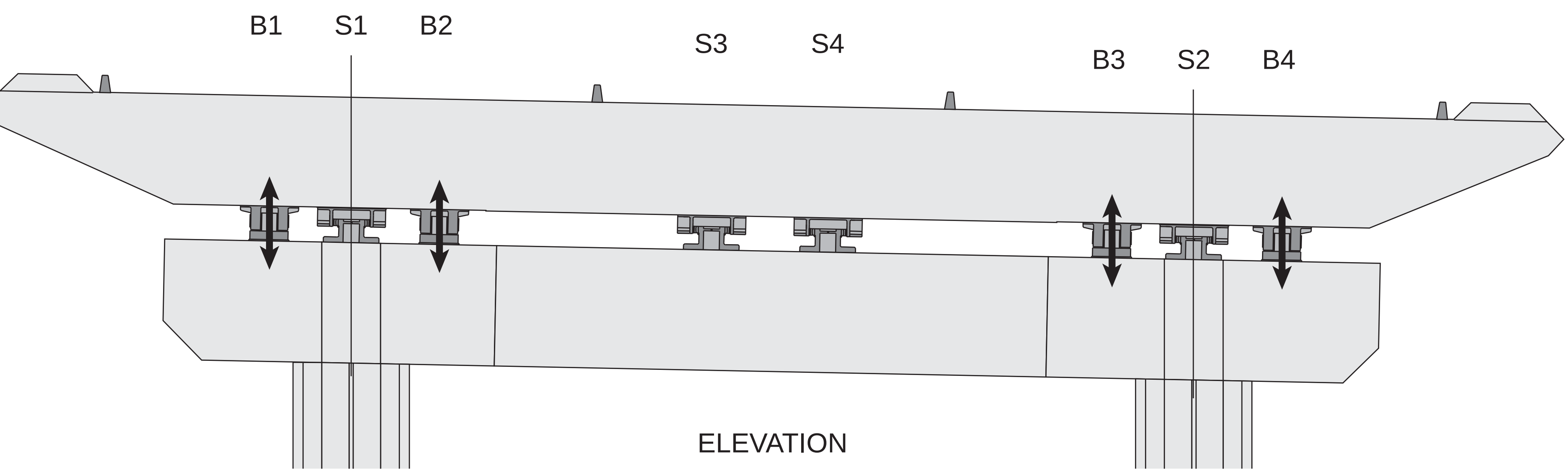
(Force Resistance Of Shear Keys And Bearings)

Direction of applied force that element can resist:





PLAN



ELEVATION

LOAD PATH C — INTERIM SHIMMING OF BEARINGS / S1 & S2 SHEAR KEY DISCOUNTED

(Force Resistance Of Shear Keys And Bearings)

Direction of applied force that element can resist:

Uplift

Transverse

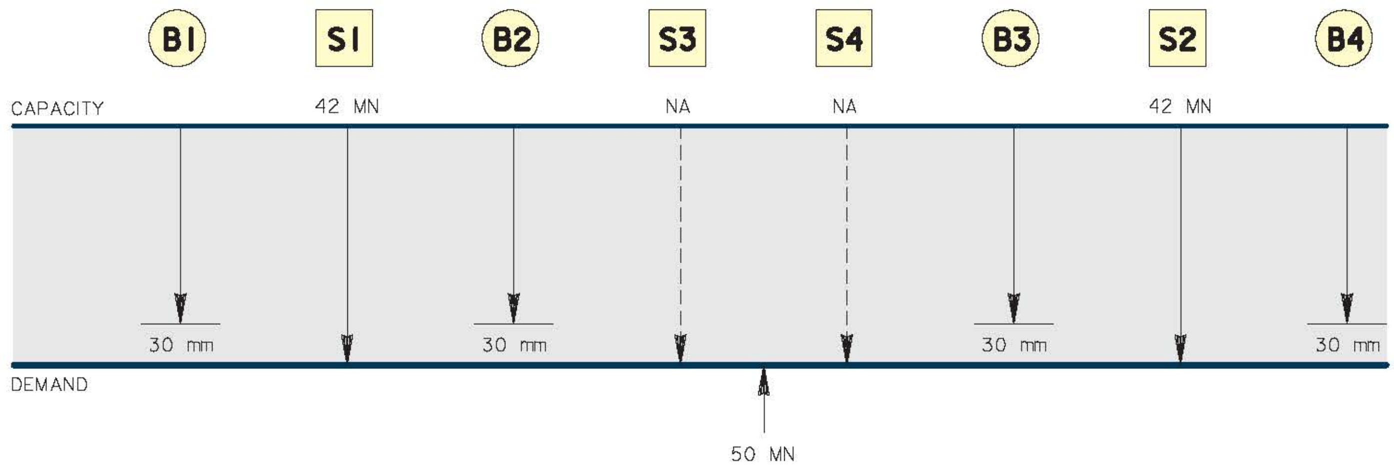
Longitudinal

LONGITUDINAL

(DEMANDS & CAPACITIES)

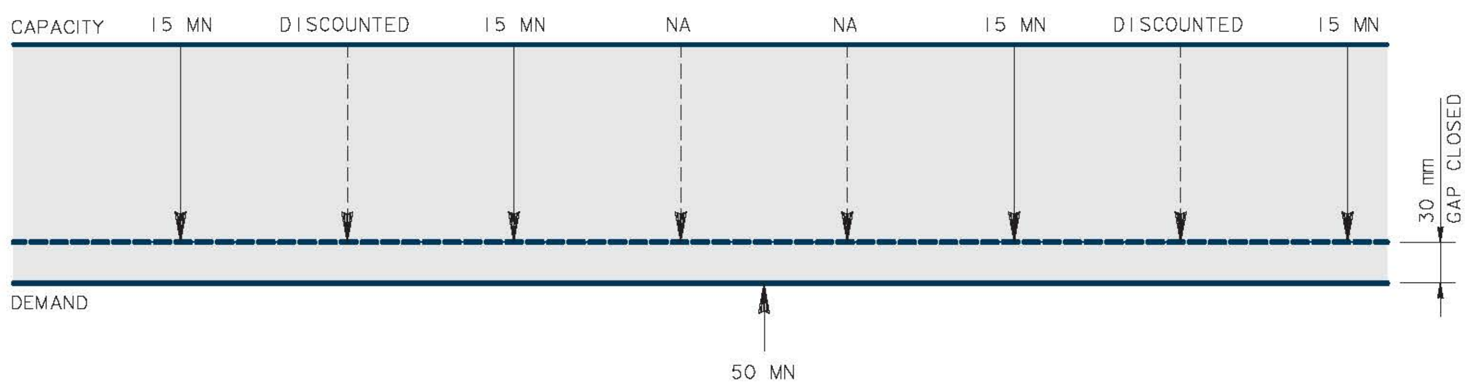
LOAD PATH (A)

- AS DESIGNED NOMINAL



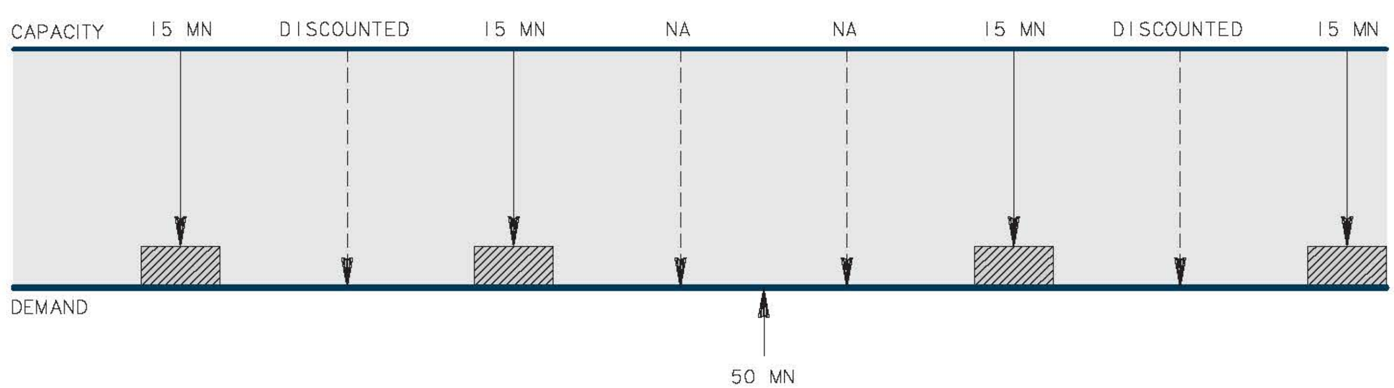
LOAD PATH (B)

- AS DESIGNED GAP CLOSED
- S1, S2 DISCOUNTED
- S3, S4 NOT ENGAGED



LOAD PATH (C)

- INTERIM FIX
- GAP CLOSED BY SHIMMING
- S1, S2 DISCOUNTED
- S3, S4 NOT ENGAGED

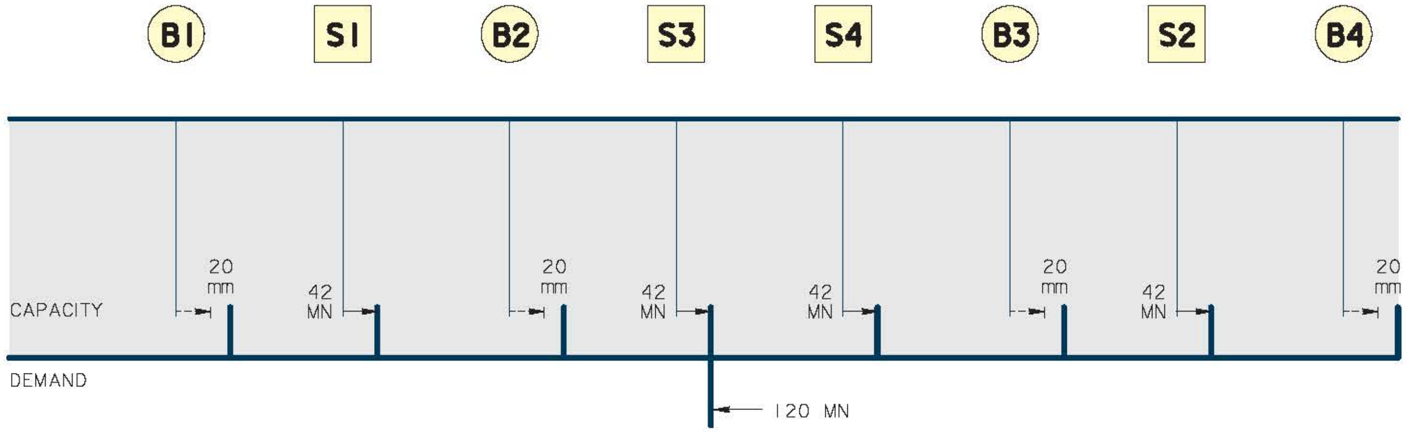


TRANSVERSE

(DEMANDS & CAPACITIES)

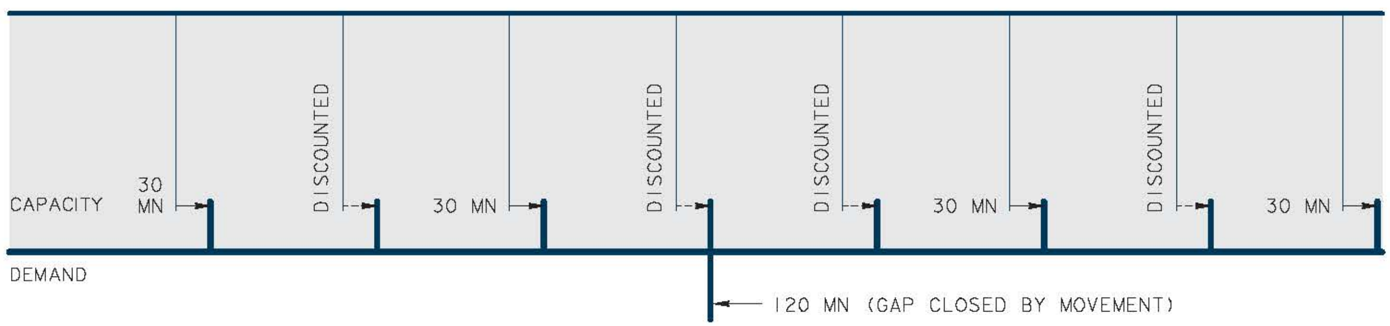
LOAD PATH A

– AS DESIGNED NOMINAL



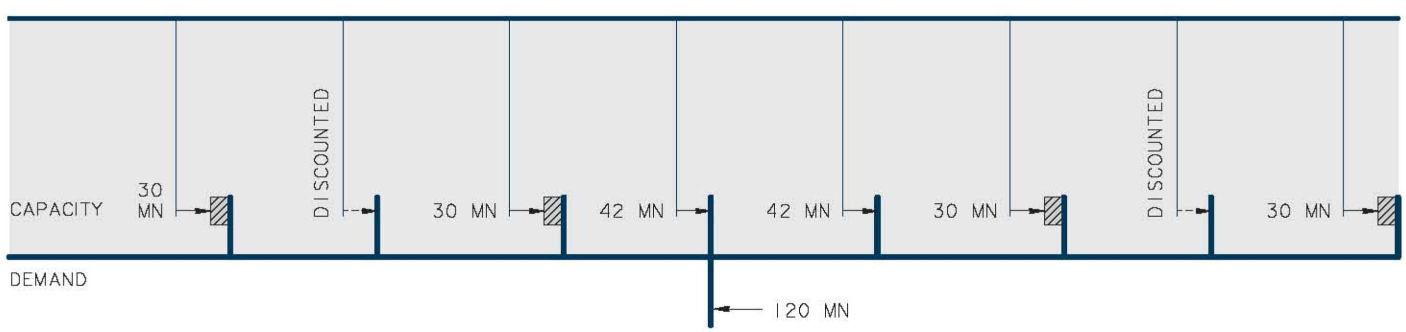
LOAD PATH B

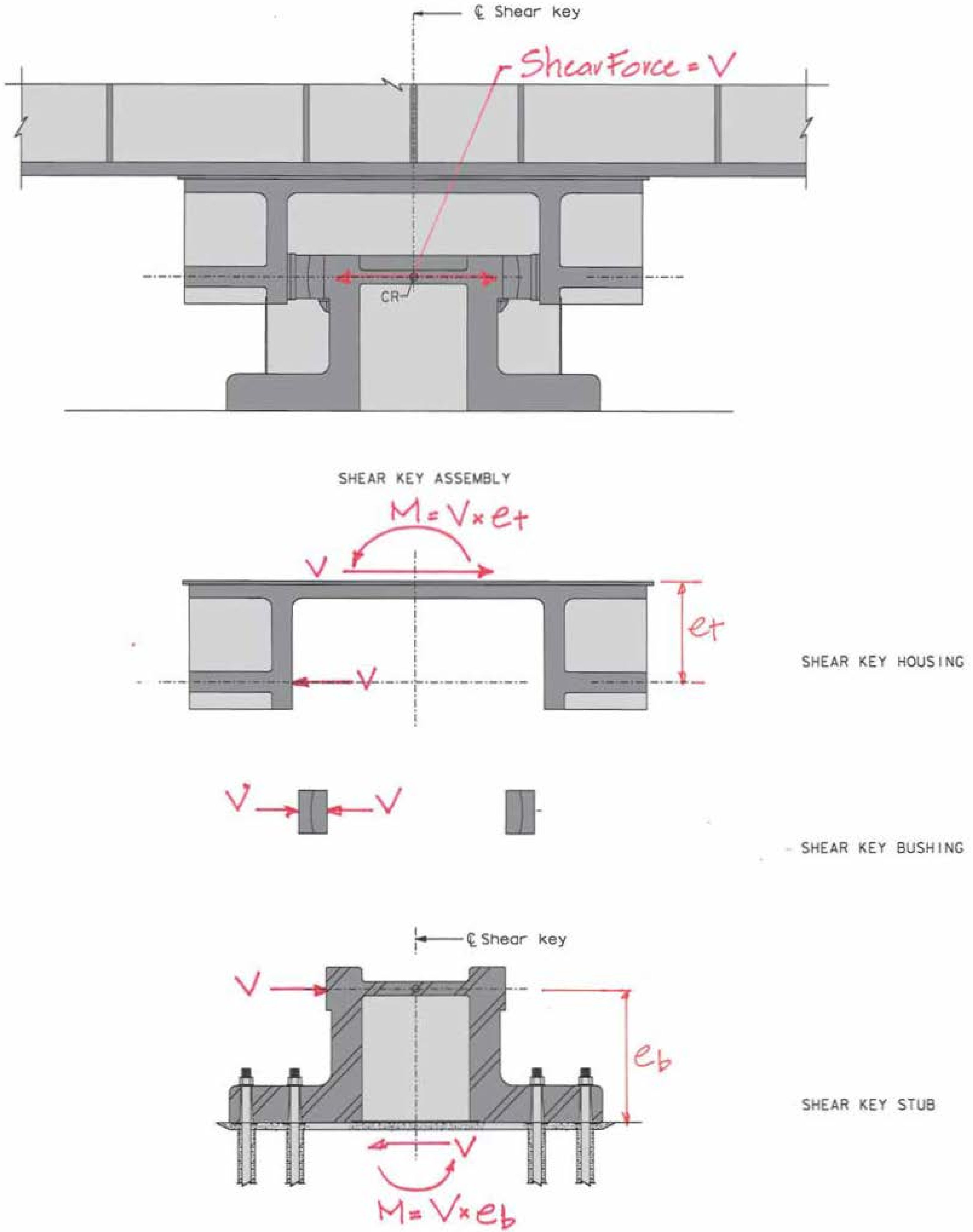
– AS DESIGNED GAP CLOSED
– S1, S2, S3 & S4 DISCOUNTED



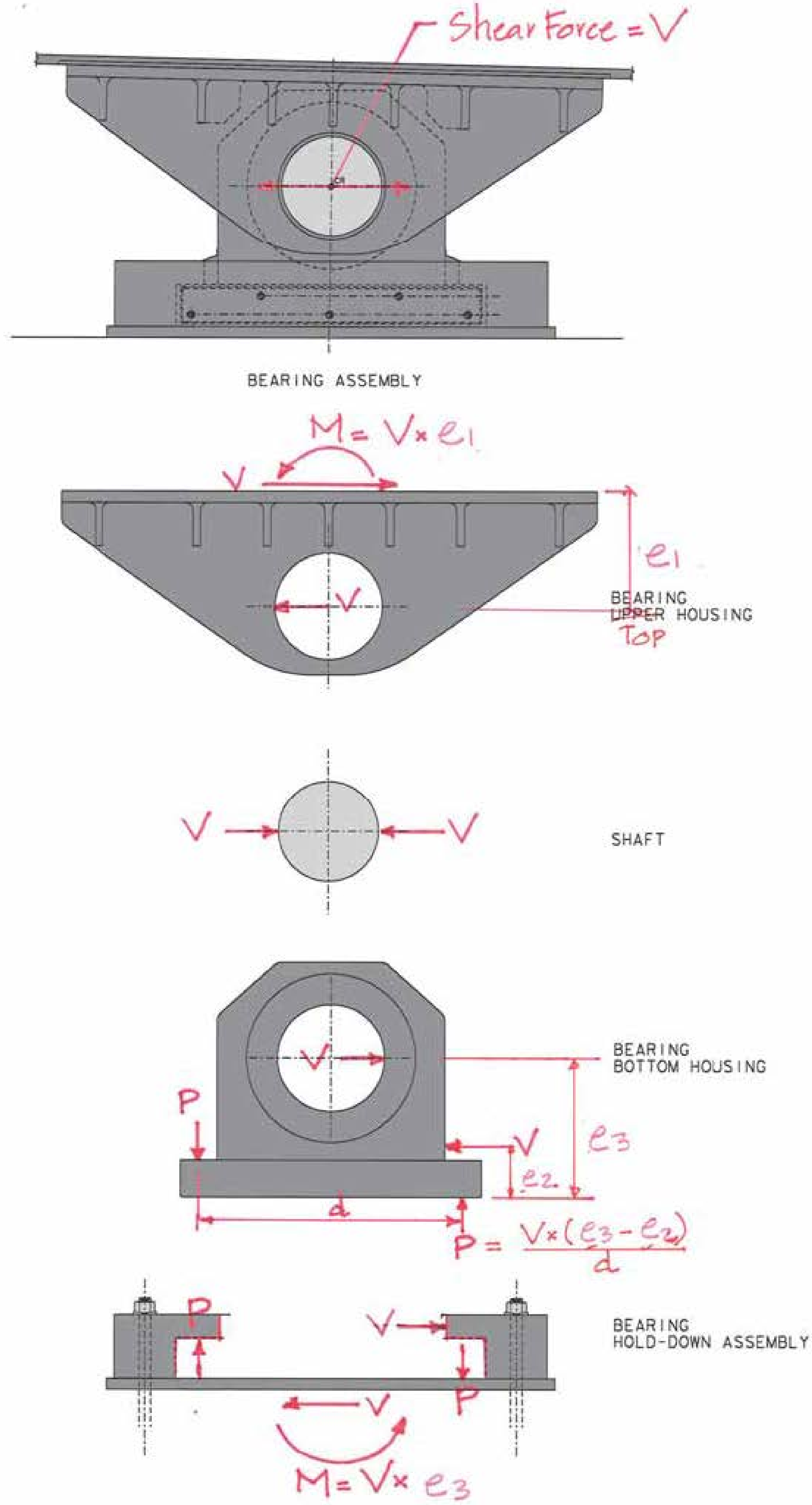
LOAD PATH C

– INTERIM FIX
– GAP CLOSED BY SHIMMING
– S1, S2 DISCOUNTED

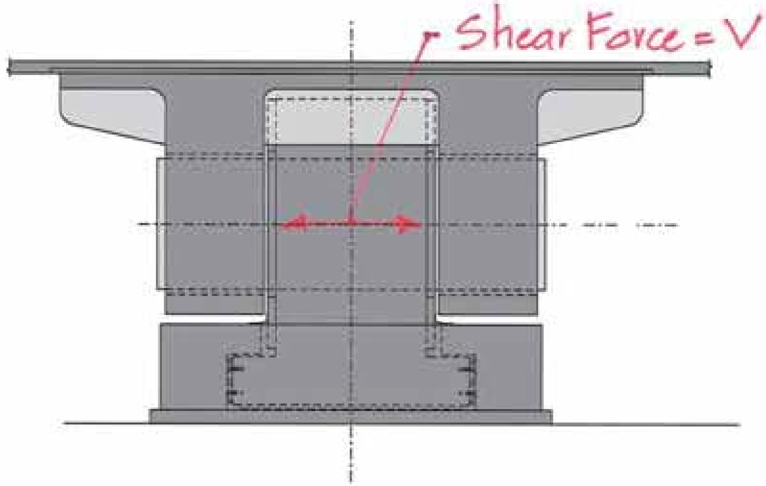


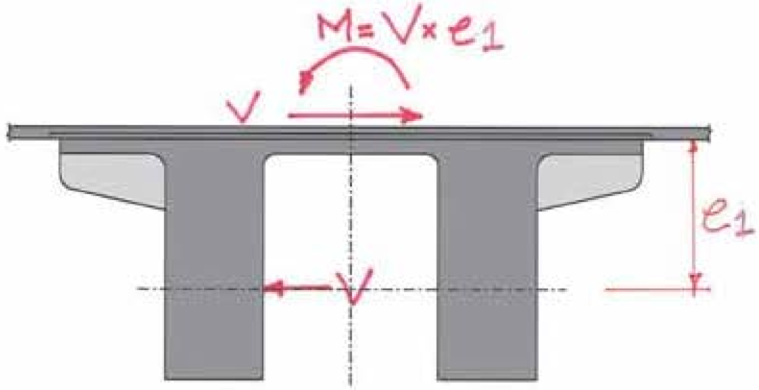


LOAD THROUGH SHEAR KEY

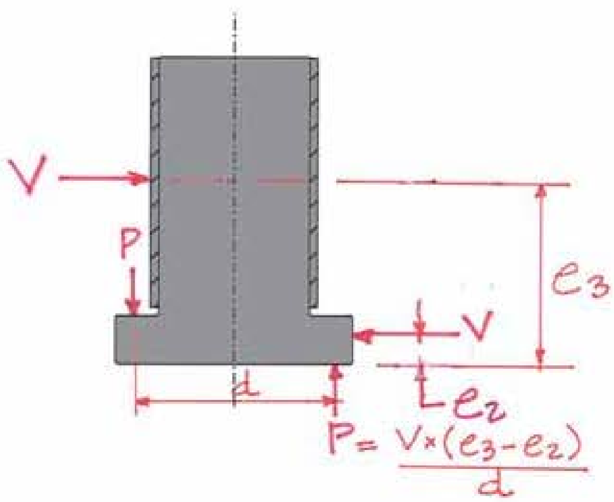


LOAD THROUGH BEARING – LONGITUDINAL SHEAR

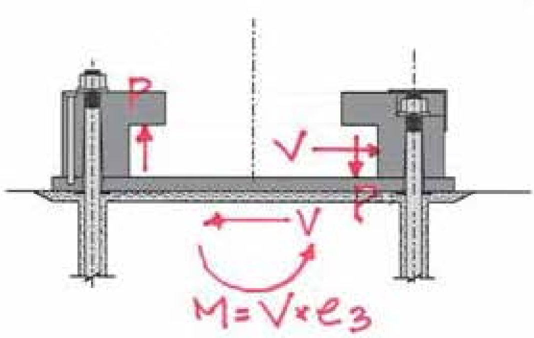




BEARING TOP HOUSING

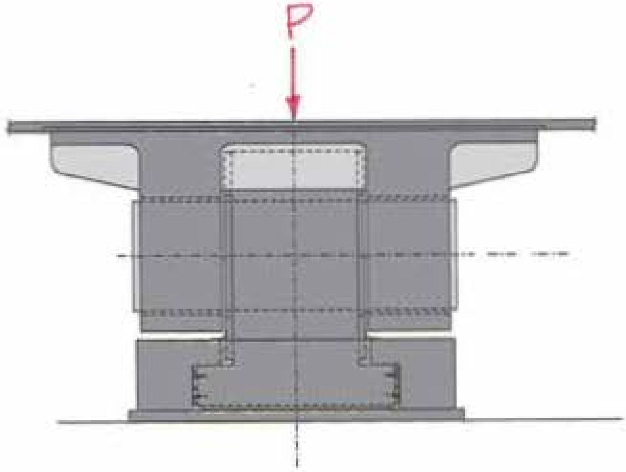


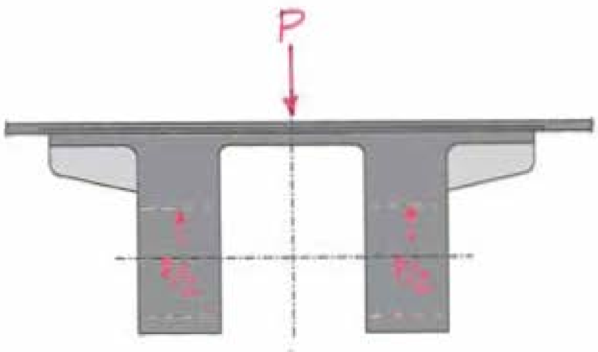
BEARING BOTTOM HOUSING



BEARING HOLD-DOWN ASSEMBLY

LOAD THROUGH BEARING – TRANSVERSE SHEAR





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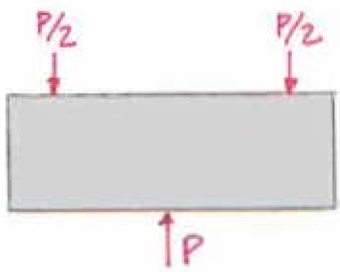
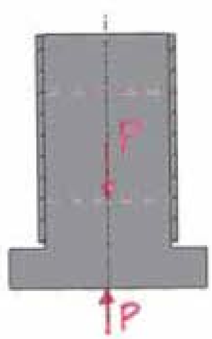
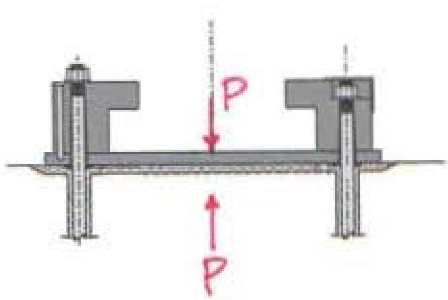


SHAFT

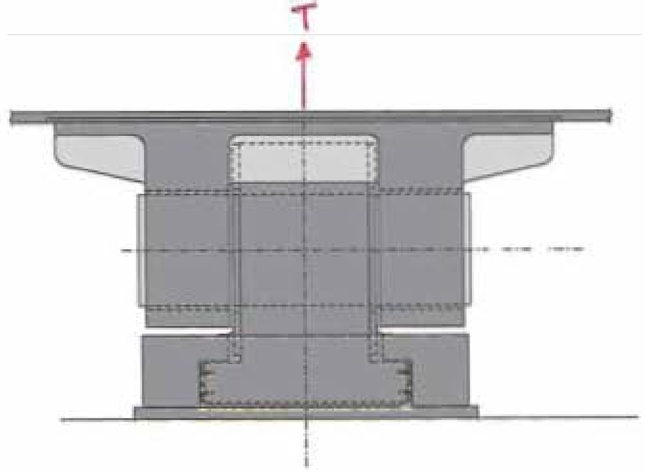


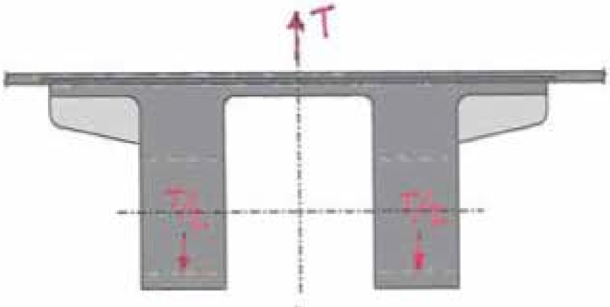
BEARING BOTTOM HOUSING



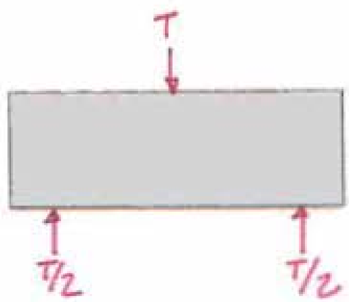
BEARING HOLD-DOWN ASSEMBLY

LOAD THROUGH BEARING – COMPRESSION

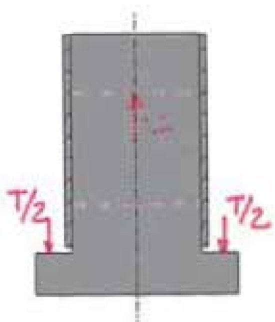




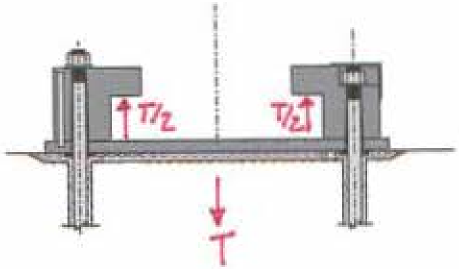
BEARING TOP HOUSING



SHAFT



BEARING BOTTOM HOUSING



BEARING HOLD-DOWN ASSEMBLY

LOAD THROUGH BEARING - UPLIFT

CCO No. 331 – E2 Bearing Shimming Details (883S1 of 1204)

DIST.	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
04	SF	80	13.2/13.9	883SI	1204

Caltrans Metric

REGISTERED ENGINEER - CIVIL

07-03-13

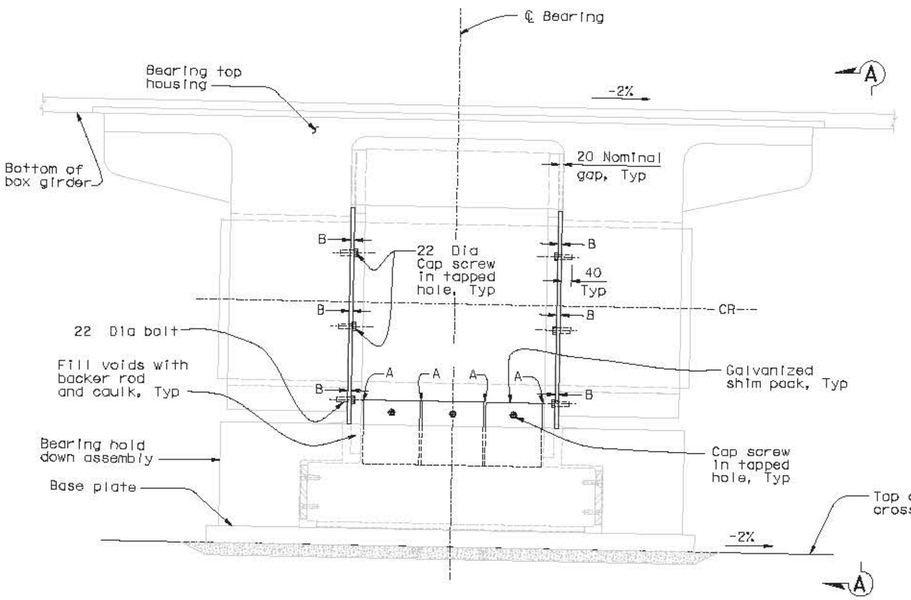
PLANS APPROVAL DATE

The State of California or its officers or agents shall not be responsible for the accuracy or completeness of electronic copies of this plan sheet.

REGISTERED PROFESSIONAL ENGINEER Marwan N. Nader No. C 054426 Exp. 12/31/13 CIVIL STATE OF CALIFORNIA

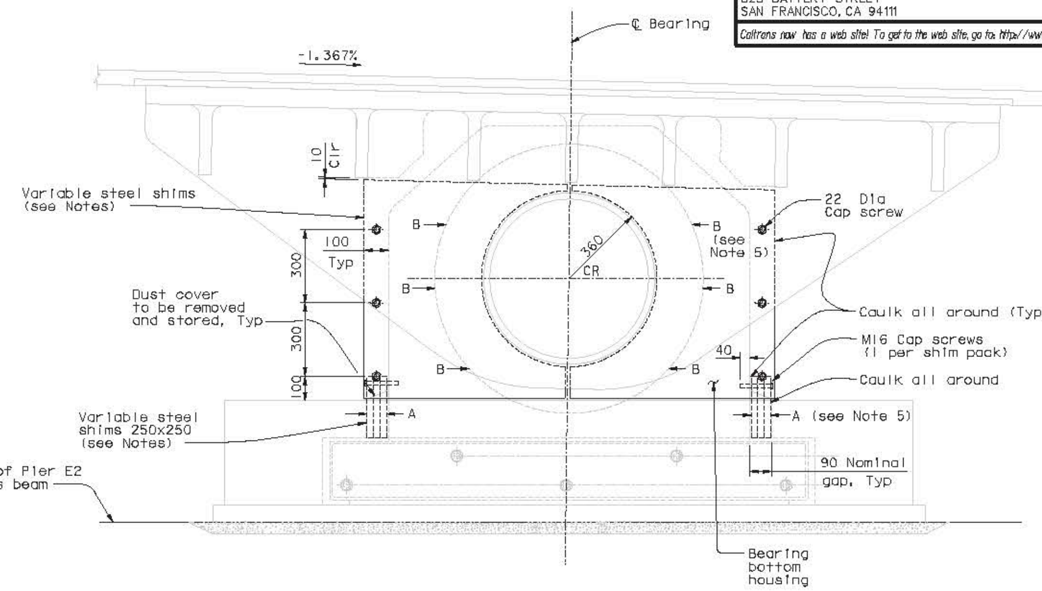
T.Y. LIN / MOFFATT & NICHOL
825 BATTERY STREET
SAN FRANCISCO, CA 94111

Caltrans now has a web site! To get to the web site, go to: http://www.dot.ca.gov



DETAIL OF BEARING ASSEMBLY

1:10



VIEW A-A

1:10

INSTALLATION SEQUENCE FOR BEARING SHIMS:

NOTE: Shimming shall be completed prior to SSO.

1. On one OBG (EB or WB), install longitudinal and transverse shimming on both bearings.
2. On the same OBG and for one temporary bearing at a time, transfer the load to the permanent bearing and remove temporary bearing shimming. Repeat for both temporary bearings.
3. Repeat Steps 1 & 2 for the remaining OBG.

NOTES:

1. Shims shall be galvanized.
2. Each shim shall be PTFE coated on one side.
3. Provide for each shim pack 4 shims tapered at 1/40 maximum, as required per survey.
4. Shim tight to within 2 mm, total of all plies.
5. Variable shims shall be fabricated and provided based on field measurements of gaps in the bearing assembly. Gap "A" in the lower housing is 90 mm nominal. Gap "B" in the upper housing is 20 mm nominal.

E2 BEARING SHIMMING

CONTRACT CHANGE ORDER NO. _____
SHEET ____ OF ____

REQUESTS FOR INFORMATION NOT ADDRESSED IN THIS CCO REMAIN IN FORCE

ALL DIMENSIONS ARE IN MILLIMETERS UNLESS OTHERWISE SHOWN

A. Akinsanya / W. Long
DESIGN OVERSIGHT
SIGN OFF DATE 07/03/13

MARK	DATE	DESCRIPTIONS	BY	CH'D	CCO#
1	07/03/13	SHIMMING OF E2 BEARINGS	GB	MN	331

REVISIONS

	BY	CHECKED
DESIGN	G. Baker	N. Vo
DETAILS	G. Baker	N. Vo
QUANTITIES	G. Baker	N. Vo

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

M. Nader
PROJECT ENGINEER

BRIDGE NO. 34-0006L/R
KILOMETER POST 13.2/13.9

SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT
SELF-ANCHORED SUSPENSION BRIDGE
(SUPERSTRUCTURE & TOWER)

PIER E2 BEARING DETAILS NO. 1A

Rev. Date: 5-18-98

ORIGINAL SCALE IN MILLIMETERS FOR REDUCED PLANS 0 10 20 30 40 50 60 70 80 90 100

CU 04
EA 0120F1

DISREGARD PRINTS BEARING EARLIER REVISION DATES 07/03/13

REVISION DATES (PRELIMINARY STAGE ONLY)

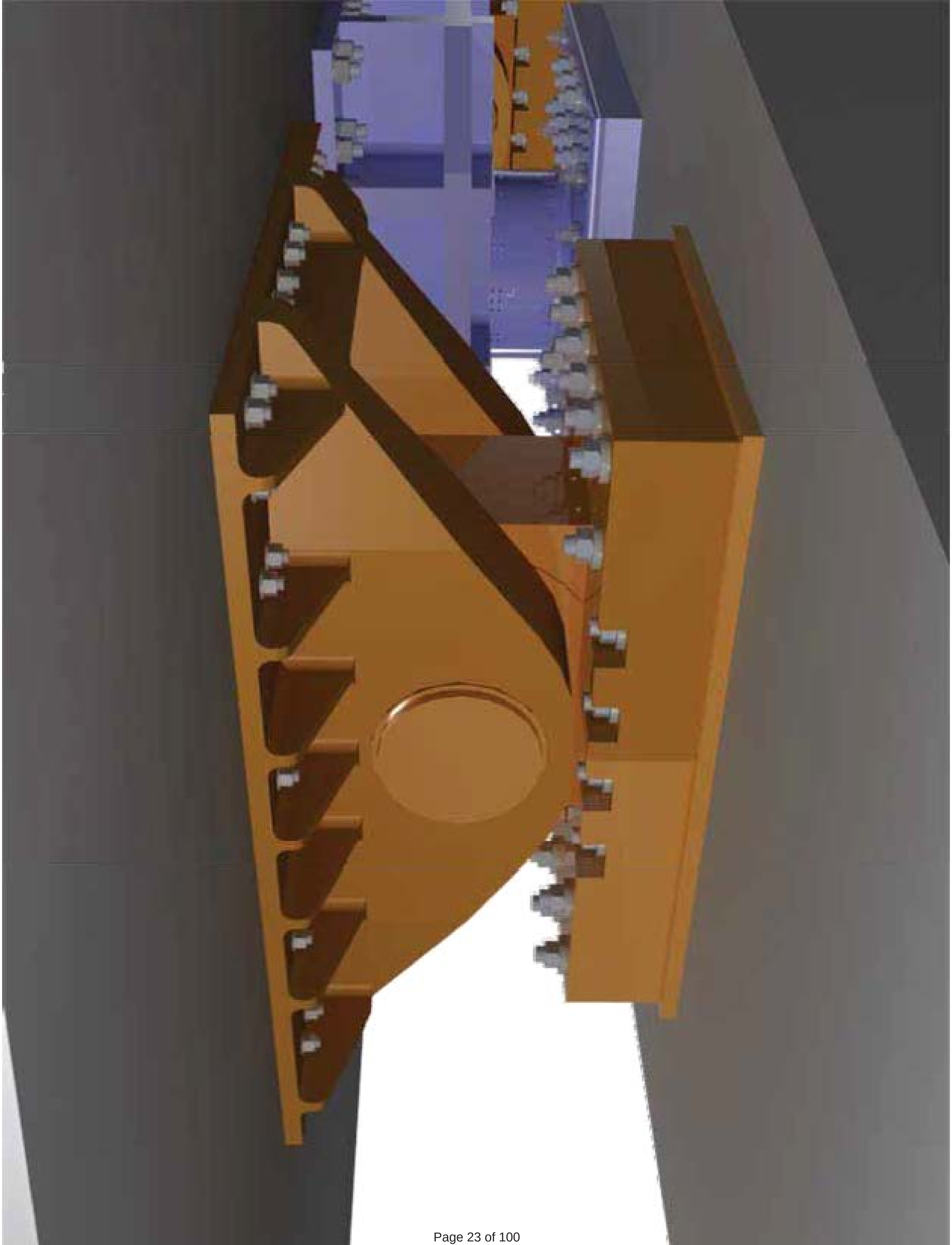
SHEET 466SI

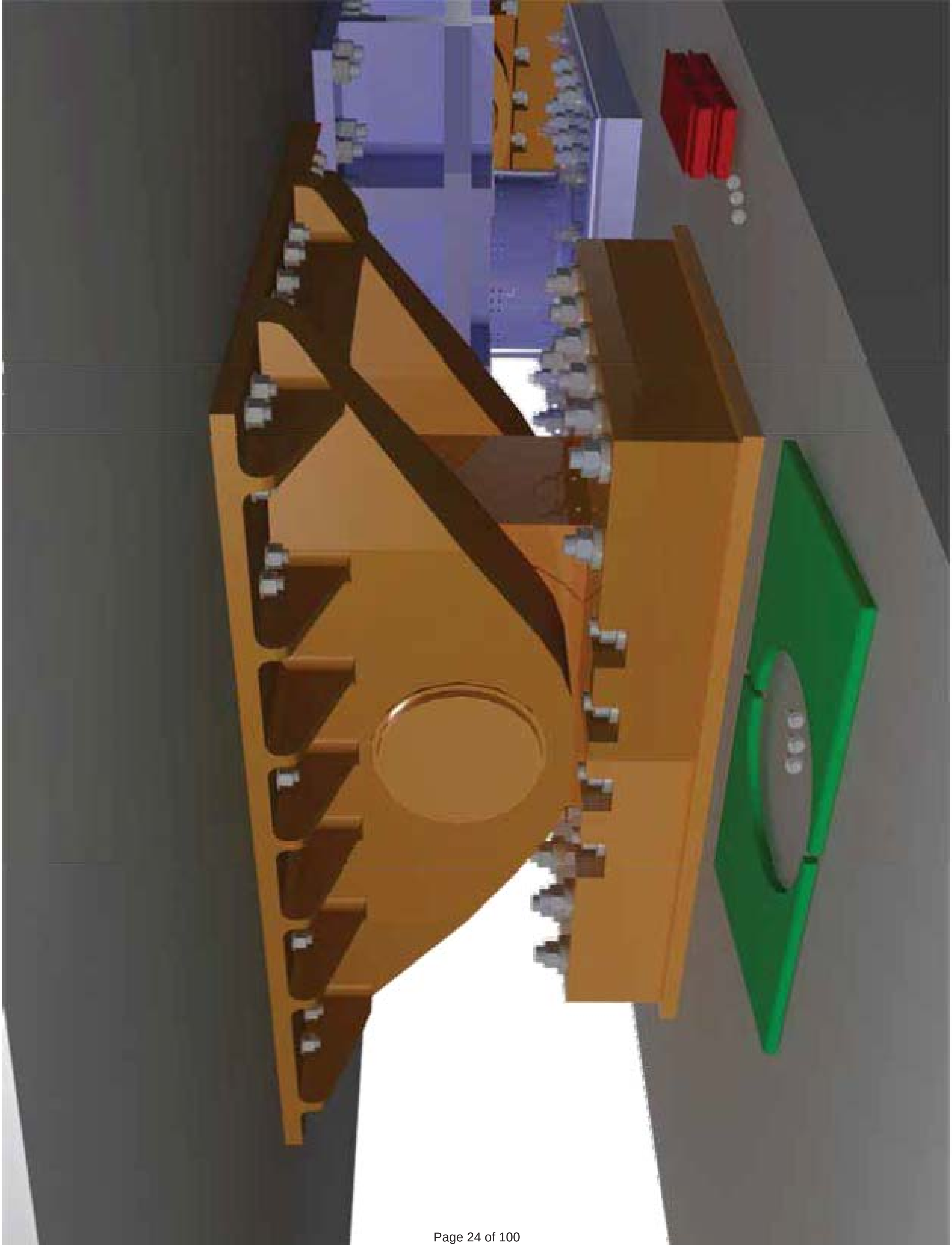
FILE => L:\BB\04-0120F1\SAS\Contract Plans and CCO\CCO\CCO#331\1-Original 07-01-13\DGN\akbrg01a.dgn

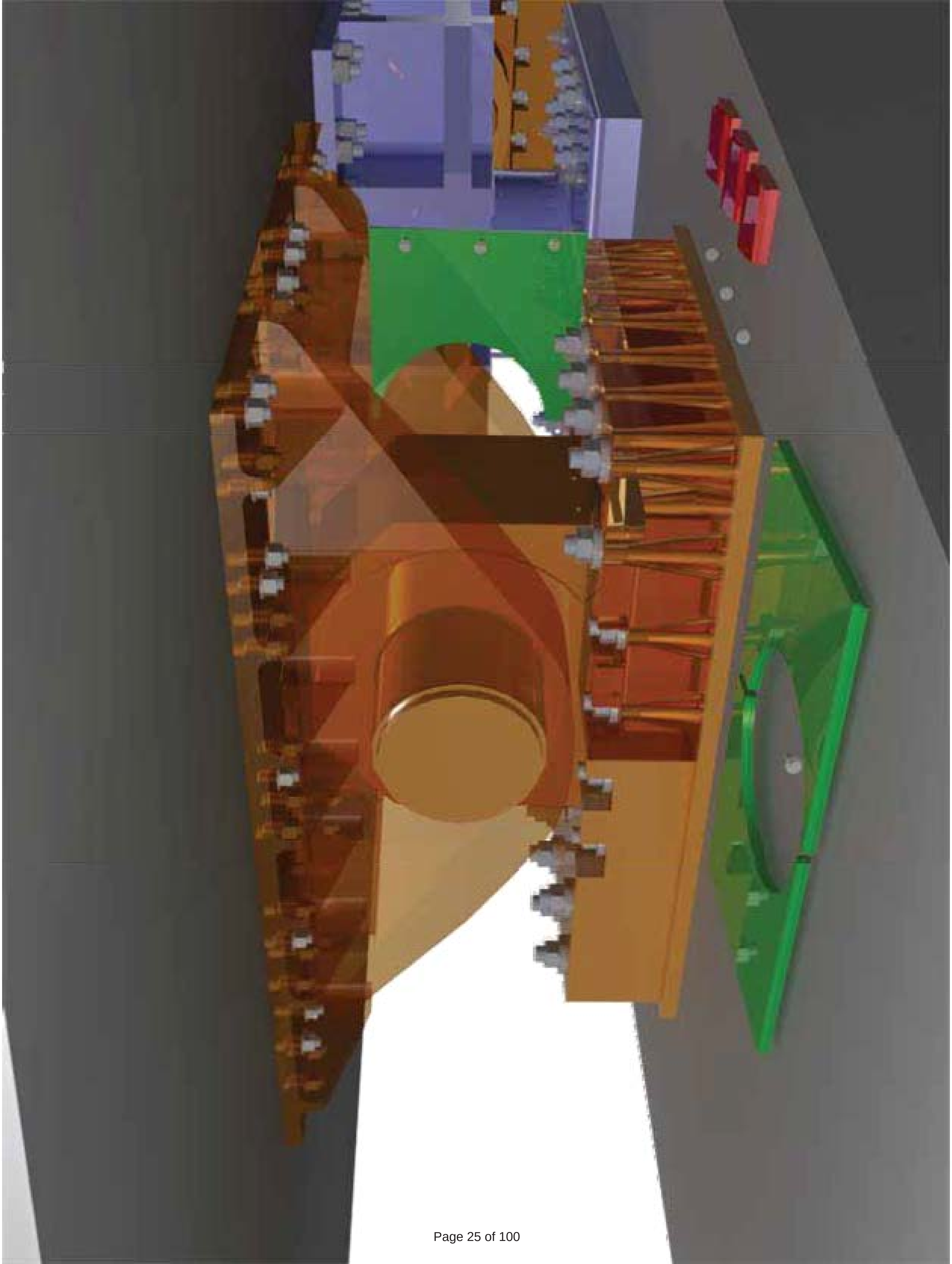
USERNAME => driivera DATE PLOTTED => 03 JUL 2013 TIME PLOTTED => 18:19:03

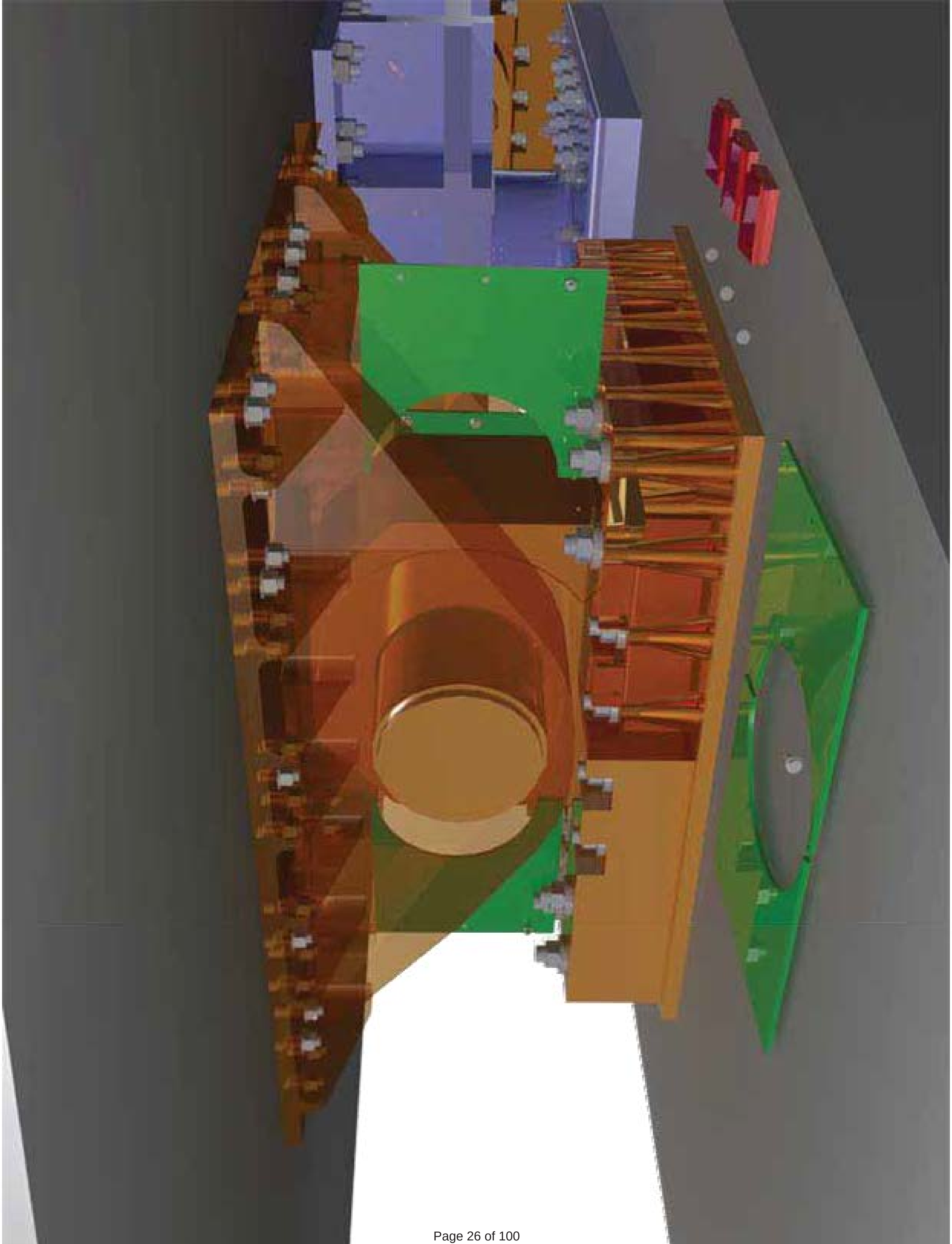
100 % PS&E

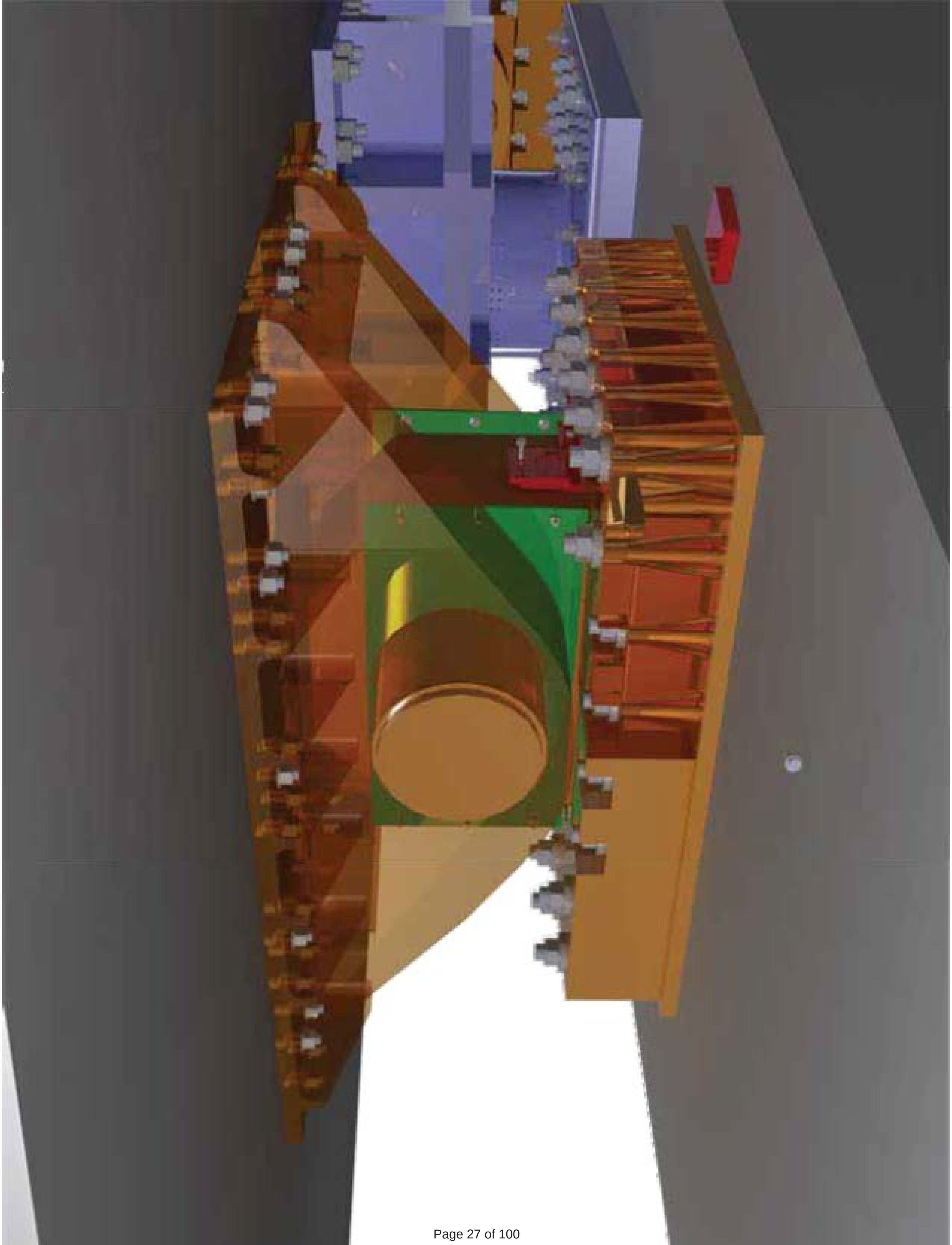
Shim Installation Sequence

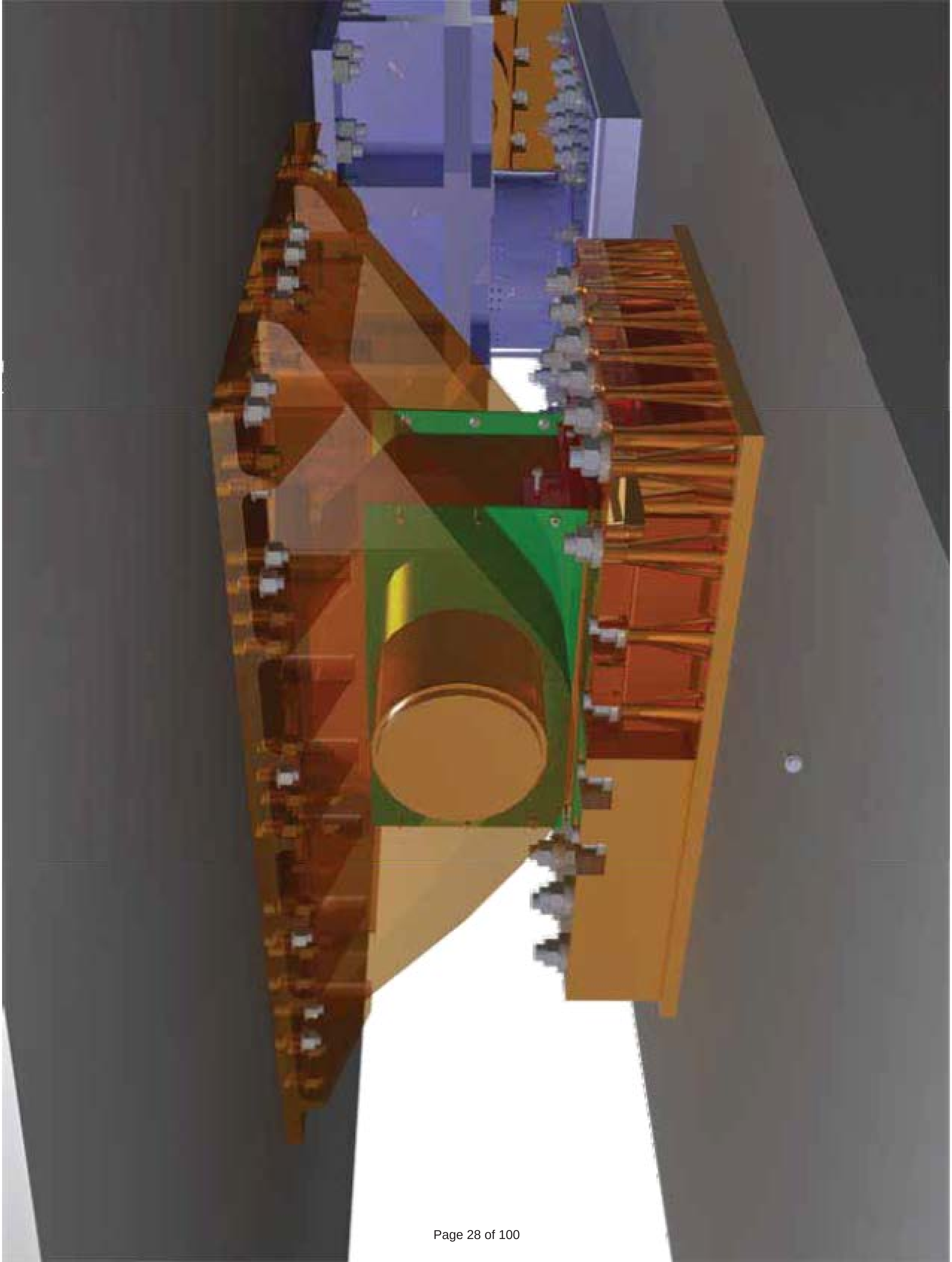


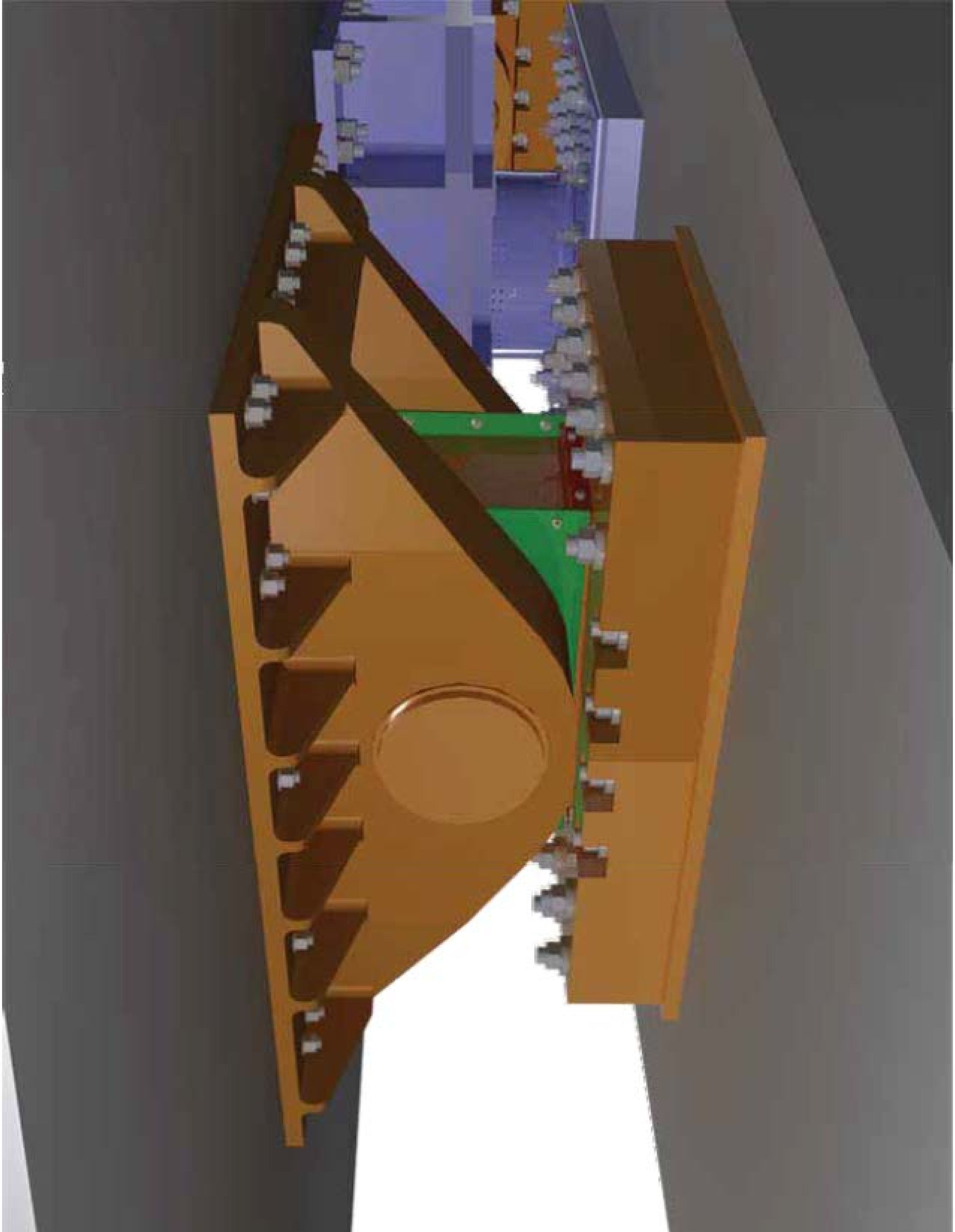












To: Toll Bridge Oversight Committee (TBPOC)
From: Toll Bridge Seismic Safety Peer Review Panel (SSPRP)

August 8, 2013

Ref: Seismic Safety and Opening of the New San Francisco Oakland Bay Bridge

Dear TBPOC:

We are writing to you to restate our opinion that the opening of the New San Francisco Oakland Bay Bridge East Bay Spans should not be delayed once seismic safety has been achieved. This has been and is the opinion of the Caltrans SAB and the Toll Bridge SSPRP from the very beginning. We have already experienced significant delays for this important seismic safety project due to political posturing and inquiries, prompted by technical misinformation. To date we are fortunate that we did not have a repetition of a Loma Prieta type event or an even bigger earthquake in the Bay Area, an event that is almost certain to occur, we just don't know when.

The existing East Bay Spans of the Bay Bridge, while upgraded and retrofitted following Loma Prieta, still do not provide the desired and even mandated level of life safety for even ordinary bridges (exactly the reason why we are building the new bridge) and should be decommissioned ASAP, namely as soon as traffic can be moved to the new bridge safely, both in terms of traffic operations and earthquakes. The new bridge, even with two of the shear keys on bent E-2 not yet functioning, has already achieved a level of seismic safety that exceeds that of the old bridge by at least a factor of two. Actually, based on all analyses provided to date, the new bridge (with shims installed as discussed below) has met the design intent - namely to withstand the 1,500 year event with minimal damage that will allow traffic operations after inspection and minor repairs as desired for a life line bridge.

In most other bridges the seismic loads are carried by the bearings and the shear keys are the secondary system that will catch the bridge once the bearings have failed. The New Bay Bridge is different. For the New Bay Bridge the shear key system on E-2 was conceived and designed to carry the entire lateral load. To achieve this, the rocker bearings have a gap on both sides so that the transverse loads are not transmitted through the bearings. This gap also allows for improved rotation, easier inspection, and reduced maintenance of the bearings over the 150 year design life of the bridge.

Currently, shear keys S1 and S2 are not complete and cannot carry lateral loads. However, in case of an earthquake the lateral load can be carried by the center shear keys S3 and S4 on the cross beam of bent E-2 and the four bearings as long as the gaps in the bearings are closed and the bearings act jointly with shear keys S3 and S4. This short-term interim solution has been rationally evaluated by the designers and Caltrans, including FEM analysis of the bearing upper and lower housings for seismic loads as well as the bearing hold down assemblies (see Seismic Evaluation of SAS at E-2 Pier prior to completion of shear Keys S1 & S2, July 15, 2013).

The closing of the gaps is accomplished through the installation of shims which are Teflon coated machined steel plates that still allow bearing rotation but no significant lateral movement in the bearings. The only drawback is that inspection and maintenance of the bearings is somewhat more difficult during the short time the shims are in place. This drawback is more than offset by the protection that will be provided to the bearings by the shims should a major earthquake occur before shear keys S1 and S2 are retrofitted. We understand that the shims are manufactured, were delivered to the site, and can be installed in one week.

With the design level of seismic safety achieved with shims in place, there is no reason or justification to delay the opening of the new bridge. This way we can provide seismic safety for the general public at the earliest possible time. As long as we are ready with all other traffic and seismic related safety and functionality issues, there is no need to wait until December (or whenever the outer shear keys S1 and S2 on E-2 are completed) to open the bridge. The New San Francisco Oakland Bay Bridge East Bay Spans should be opened for traffic on Labor Day, as originally planned.

In the opinion of the SSPRP, the State is taking on a higher risk in delaying the opening of New Bridge than in opening the New Bridge at the earliest possible date while continuing with the retrofit of shear keys S1 and S2.

For the SSPRP
Sincerely Yours

Frieder Seible, chair, SSPRP

U.S. Department of Transportation
Federal Highway Administration

California Division

650 Capitol Mall, Suite 4-100
Sacramento, CA 95814
(916) 498-5001
(916) 498-5008 (fax)

August 9, 2013

In Reply Refer To:
HDA-CA

Mr. Steve Heminger
Chair, Toll Bridge Program Oversight Committee
Department of Transportation
Office of the Director
1120 N Street
P.O. Box 942873
Sacramento, CA 94273

Subject: FHWA's Response to Review Requests of the San Francisco-Oakland Bay Bridge

Dear Mr. ~~Heminger~~: Steve

This letter is in response to two reviews that the Toll Bridge Program Oversight Committee (TBPOC) asked the Federal Highway Administration (FHWA) to perform of the San Francisco-Oakland Bay Bridge East Span (SFOBB) Seismic Replacement Project. One review was for the disposition of the Galvanized ASTM 354 Grade BD bolts/rods, and the other for the Pier E2 interim installation of bearing shims to allow opening the bridge while the shear key retrofit progressed.

We have concluded both reviews and agree with the approaches. We concur with the disposition of the bolts/rods and the recommended course of action described in the TBPOC's final report dated July 8, 2013, as shown in Table ES-2 and subsequent language on pages ES-16 and 17. FHWA also agrees with the strategy for an interim shim proposal to temporarily restore the shear capacity lost, allowing the bridge to be open to traffic prior to and during the retrofit of the S1 and S2 shear keys. In addition, FHWA has reviewed and agreed that the post-tensioned retrofit of the Pier E2 S1 and S2 shear keys will adequately replace the function of the failed anchor rods. The following paragraphs describe our review observations:

The first request, made on May 8, 2013, asked for "FHWA to conduct an independent review of our (TBPOC) findings and recommendations concerning the galvanized high strength bolts…". These high strength bolts or rods consist of Galvanized ASTM A354 Grade BD material and are located on the Self-Anchored Suspension (SAS) span totaling 2,306 in number and listed in 17 different locations. The TBPOC's July 8, 2013 final report classified the bolts into the following categories:

1. Rods whose clamping capacity is to be replaced before opening the bridge to traffic;
2. Rods that are to be replaced after opening the bridge, as a precautionary measure to address concerns of longer-term stress corrosion;

3. Rods that are subject to mitigating actions, such as reduced tension, dehumidification or other corrosion protection systems; and
4. Rods that are acceptable for use, will meet performance expectations, and will undergo a regular inspection schedule.

The FHWA's independent review to determine the disposition of the bolts/rods consisted of:

- Site visits to see first-hand where the different categories of bolt/rods were located, with particular focus on Pier E2 where over one-half of the bolts/rods are located (locations 1-6), the tower base (locations 12 -13), the suspension cable anchorages (location 7), the east saddle anchor and tie rods (locations 14 and 15), and the cable band strong back anchor rods (location 16).
- Review of contract documentation, including but not limited to special provisions, quality assurance audit documentation of fabricators and sub-contractors, material acceptance testing (including chemistry, average hardness, and tensile strengths), heat treating, cleaning, and galvanization processes, contract drawings, grouting procedures, and bolt/rod installation and tensioning.
- Review of testing documentation post failure of the Pier E2 S1 and S2 shear key masonry plate anchor rods. This included interviews of project staff and experts as well as presentations of tests performed with results of those tests, including comparisons between the rods supplied in 2008 and rods supplied in 2010. Tests included Rockwell hardness values across ends of the bolts/rods, Charpy V-notch toughness tests, tensile capacity tests, and metallurgical evaluations of failure planes.
- Review of the retrofit of the Pier E2, S1 and S2 shear key masonry plate anchor rod retrofit plan sheets and special provisions. The team also visited the site to view the preparatory work of the pier cap to accommodate the retrofit, interviewed the designer of record and project staff to discuss our observations, and participated in technical update discussions. We discussed the placement of the concrete and mix type as well as PL2 corrosion protection practices. We also discussed post-tensioning stages and strength requirements prior to post-tensioning, including projected losses at release with the project engineer and designer of record. In conclusion, the team was satisfied with the review material and the resulting discussions regarding the retrofit and concurs with this approach as a solution to replacement of the failed anchor rods.

Throughout our review of the Galvanized A354 Grade BD bolts/rods, we asked many questions, all of which were answered. Many of our discussions resulted in tasks to perform during the next few weeks as the bridge nears completion. For example, project staff and consulting engineers continue to perform testing, such as the Townsend and baseline ultrasonic testing, which can be used to develop and implement long-term monitoring, maintenance, and replacement strategies. One of our observations resulted in a recommendation that project staff begin dehumidifying applicable bolt/rod locations as soon as practical.

Your second request, dated July 12, 2013, asks FHWA to "conduct an independent review of the Seismic Safety Peer Review Panel (SSPRP) proposal to shim the bearings at Pier E2" of the self-anchoring suspension span.

FHWA's review included an assessment of the proposal documentation, a site visit to see first-hand where bearing translation would be limited to engage seismic forces through use of translation "limiters" (also referred to as "shims"), and technical discussions with project engineers and the designer of record.

The FHWA review team was impressed with the level of expertise used to fashion this interim means of limiting the movement of the bearings so they engage and safely transfer seismic forces while the installation of the Pier E2 S1 and S2 shear key retrofit continues. Once the Pier E2 bearing movement limiters are properly installed, this interim solution will restore the capacity lost prior to and during the retrofit of the S1 and S2 shear keys and provide a comparable level of seismic performance. As such, we see no reason to delay opening the bridge to traffic prior to the shear key retrofit being completed.

We want to thank your staff, consultants, and construction workers for their professionalism during our review.

Should you have questions, please do not hesitate to contact me at (916) 498-5001 or by email at Vincent.Mammano@dot.gov.

Sincerely,

Vincent P. Mammano
Division Administrator

BUCKLAND & TAYLOR | COWI

Independent Review of the Seismic Safety Peer Review Panel Proposal to Shim the Bearings at Pier E2 of the New East Span of the San Francisco - Oakland Bay Bridge

2013 August 06 Our Ref: 2054-RPT-GEN-001-0A

Prepared by: Peter Taylor, P.E.

Reviewed by: Brian Morgenstern, P.E.

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Executive Summary

The 2013 Labour Day opening schedule for the new East Span of the San Francisco-Oakland Bay Bridge has been threatened by the premature failure of some anchor bolts in shear connectors S1 and S2 at Pier E2. These bolts are embedded into the crossbeam of Pier E2 and cannot quickly be replaced.

The Seismic Safety Peer Review Panel has proposed to temporarily shim bearings B1, B2 and B3, B4, which respectively are close to shear connectors S1 and S2, thereby converting the bearings into shear connectors offering load paths equivalent to S1 and S2 for the 1500 year return period design seismic event.

Dr. Peter R. Taylor, P.E. of Buckland & Taylor has been invited to conduct this independent review of the engineering analysis and strategy of the “shim concept”, which includes opening the new east span to traffic once the bearing shims are installed, before the shear key retrofit is complete.

The review has comprised a one day site visit (which included lengthy discussions with Dr. Brian Maroney, P.E., Toll Bridge Seismic Retrofit Program Chief Bridge Engineer, James Duxbury, P.E., Senior Associate of the Design Team of T.Y. Lin – Moffatt & Nicol and a number of other members of the Owners Design and Construction Staff), a review of the Report on the bolt failures[1] and the Report on the Seismic Evaluation of Pier E2[2] and other documents plus exchanges of written questions and answers.

In order to fully understand the evolving seismic load path, the four successive stages of seismic connectivity at Pier E2 were identified and are reported in Section 4 of this review document. The capacities and functions of the critical elements were then calibrated against the seismic strength and performance demands, permitting conclusions to be drawn regarding the validity of the shim concept.

Strategic decisions taken early in the design phase of the project, such as designing for 140% of the maximum seismic demand at this key support, and creating the seismic connectivity through a family of eight redundant connections, created a favourable environment permitting consideration of temporary alternative load paths around a problem area, such as proposed by the shim concept. Consequently, this simple intervention is able to create alternative temporary load paths with seismic load transmission capacity in excess of the 1500 year return period seismic demands. The shim concept is simple, effective, economical and without any significant impairments to the seismic performance of the bridge.

The Seismic Safety Peer Review Panel is to be congratulated for identifying this effective strategy.

It is recommended that the shim concept be installed at the earliest opportunity because it restores “as designed” seismic capacity at Pier E2 and will provide appropriate interim seismic protection to users of the new bridge when it is opened.

Table of Contents

1 Scope of Work

The Scope of Work for this independent review has been defined as follows.

The Consultant shall conduct an independent review of the Seismic Safety Peer Review Panel proposal to shim the bearings at Pier E2 of the new east span of the San Francisco-Oakland Bay Bridge (SFOBB). Technical materials related to the bearing shims will be provided by BATA for the independent review of the engineering analysis and strategy for this proposal.

This bearing shim proposal includes opening the new east span to traffic once the bearing shims are installed, but before the shear key retrofit is complete. This proposal intends to temporarily restore shear capacity lost due to the failed bolts used to clamp down two shear keys on Pier E2. The bearings were designed to accommodate 20 mm of movement before engaging during a seismic event. The shims would lock the four bearings to engage simultaneously with the two remaining shear keys that are currently in place on the cross beam and functioning as designed.

Since the bearing shim proposal may offer the possibility of achieving seismic safety on the SFOBB prior to completion of the shear key retrofit, time is of the essence in the completion of the Consultant's review. The Consultant shall contact the BATA Project Manager if any additional information is required to conduct the review.

The services to be performed by Consultant shall consist of services requested by the Project Manager or a designated representative including, but not limited to, the following:

1.1 Task 1

Consultant shall provide an independent review of the engineering analysis and strategy as related to the proposal to shim the existing bearings at Pier E2 to achieve the seismic design requirements of the new east span of the San Francisco-Oakland Bay Bridge.

Consultant shall conduct a peer review of the suitability of the "Shim Concept" to provide sufficient seismic capacity at Pier E2 of the San Francisco-Oakland Bay Bridge to withstand the Safety Earthquake Event (SEE) forces until the shear keys are repaired. In performing the work, Consultant shall use:

i. Load demands at pier E2 from the analysis of the SEE pushover by the Engineer of Record (EOR) as summarized in “Seismic Evaluation of SAS at E2 Bent Prior to Completion of Shear Keys S1 and S2” dated July 15, 2013;

ii. Details of bearings, shear keys and cap beam at Pier E2 and the supports of shear keys and bearings in the orthotropic box girders provided by EOR; and

iii. Other calculations, plan sheets, shop drawings and engineering summaries prepared or supplied by request to the EOR as may be needed.

The zone of the SFOBB to be considered is the load at Pier E2 from the box girder bearing and shear key supports to the tops of the columns.

The following is expressly not included:

i. Review of the failures of the 2008 anchor bolts;

ii. Independent seismic analysis or evaluation other than force transfer in the zone identified above;

iii. Any capacity of the box girders or the substructure other than the force transfer zone identified above;

iv. Any evaluation of the repair of the shear keys (S1 and S2) or the resulting suitability;

v. Any evaluation of the original design of the cap beam at Pier E2 other than as may be needed to evaluate the changes to the load path caused by the temporary shimming; and

vi. Any other function of the shear keys and bearings other than transferring seismic demands in the temporary shimmed condition.

1.2 Deliverable

Independent review report.

2 Introduction

The proposal by the Seismic Safety Peer Review Panel to temporarily shim the bearings at Pier E2 appears, on initial inspection, to provide a simple, easily executed fix, which temporarily avoids reliance on the compromised shear keys S1 and S2 by utilizing an alternate load path through bearings B1, B2, B3, B4 to transmit the required proportion of the major seismic shear demands at that pier.

However, there is a readily understood tendency among engineers evaluating an unconventional way around an unexpected roadblock to see only the advantages of the detour strategy. Therefore, this report is structured to carefully look at how the seismic loads are transmitted in each of the steps in the four stages of development of the final revised design configuration at Pier E2 in order to detect if anything has been overlooked in this sequence.

These stages are:

i. the “as originally designed” seismic connection at Pier E2;

ii. the “current” seismic connection at Pier E2;

iii. the “proposed temporary” seismic connection at Pier E2; and

iv. the “revised final design” seismic connection at Pier E2.

This stage review permits the development of a familiarity with the load paths and details of the proposed “fix” and the varying functions of the key components.

The demands and capacities in the key components in stage iii are then reviewed prior to drawing conclusions about the overall strategy.

3 Global Seismic Demands at Pier E2

In accordance with the Seismic Demands for the Design Level Earthquake per the Project Specific Design Criteria, an envelope of the maximum time-history analysis responses from six different 1500-year ground motions (Safety Evaluation Earthquake) was derived. At the top of Pier E2, these Safety Evaluation Earthquake envelope demands are 50 MN in the longitudinal direction of the bridge and 120 MN in the transverse direction of the bridge.

4 Review of the Transmission of Horizontal Seismic Forces from the Bridge Superstructure into Pier E2 for Each of the Four Stages of Design Development of the Seismic Connections

4.1 Stage 1: Transmission of Horizontal Seismic Forces from the Bridge Superstructure into Pier E2, "As Originally Designed"

The design horizontal capacity of the shear keys and bearings at Pier E2 can be summarized as follows:

Table 1:

	Longitudinal Direction	Transverse Direction
Shear Keys S1 & S2	42 MN each	42 MN each
Shear Keys S3 & S4	42 MN each (20 mm Gap)	42 MN each
Bearings B1, B2, B3 & B4	15 MN each (20 mm Gap)	30 MN each (20 mm Gap)
Maximum Seismic Displacement	1.1 m*	0.4 m*
Total Required Capacity	50 MN	120 MN
Total Capacity Supplied	84 MN	168 MN

* Source: M. Nader 2013 August 07

The "as designed" transmission of horizontal seismic forces at Pier E2 is shown in Figure 1.

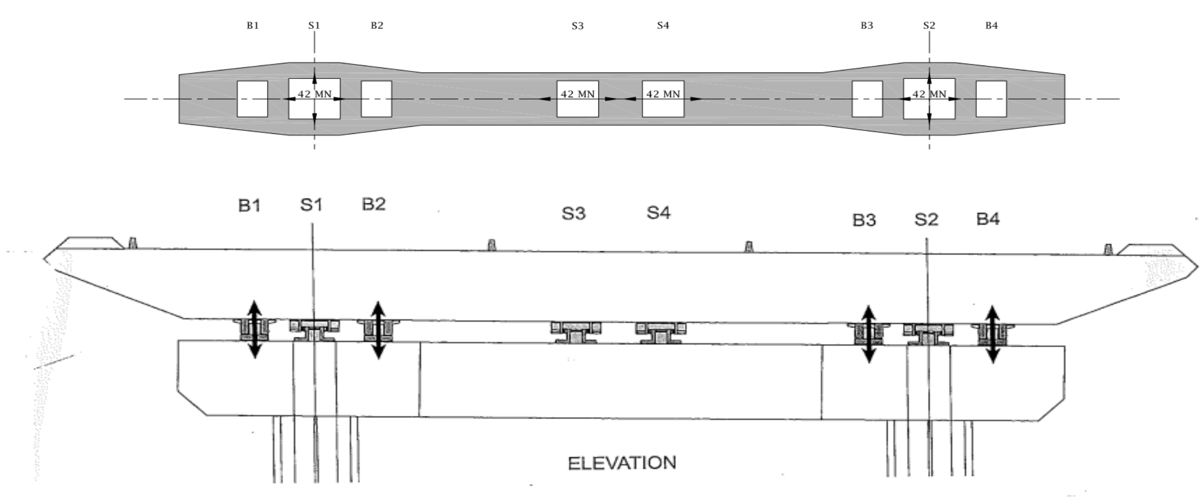

Figure 1: "As Designed" Transmission of Horizontal Seismic Forces at Pier E2

The design seismic load paths maintain the 20 mm longitudinal gaps in shear keys S3 and S4 and the 20 mm longitudinal and lateral gaps in Bearings B1, B2, B3 and B4, thereby engaging only shear keys S1 and S2 in both directions and shear keys S3 and S4 in the transverse direction. This provides a total seismic horizontal connection capacity of 84 MN longitudinally and 168 MN in the transverse direction.

The horizontal seismic demand on the shear key bases at Bent E2 comprises both shear and moment, which are resisted by the friction (coefficient 0.67) and clamping forces created by 48 bolts of 3 inch diameter (ASTM A354 Grade BD) pretensioned to 68% of their ultimate stress of 140 ksi (965 MPa) or about 3 MN per bolt.

The equivalent demand between the shear key top sections and the steel box superstructure at Pier E2 is resisted by the friction (coefficient 0.50) and clamping forces created by 80 bolts of the same diameter and pretension.

The lack of transverse clearances at the shear keys will cause some minor thermal stressing of the top beam of bent E2 and the transverse beam connecting the box girders for temperature differentials between superstructure and bent.

4.2 Stage 2: Transmission of Horizontal Seismic Forces from the Bridge Superstructure into Pier E2 under "Current Conditions"

All of the details of the "current" situation at Pier E2 are not precisely known, but it is known that at shear keys S1, S2 all existing unbroken ASTM A354 Grade BD anchor bolts have had their pretension tensile stress reduced to 0.45 f_u and the two outer longitudinal rows of anchor bolts on the bases have been detensioned and cut flush with the top of baseplate to permit installation of the steel post tension saddles.

In addition, the two temporary bearings used during erection are still in place on the crossbeam of Pier E2 (any contribution from these bearings is ignored in this discussion).

After discounting the broken anchor bolts and those removed in the saddle zone, shear key S1 has 12 functional anchor bolts out of an original 48 and S2 has 16. These anchor bolts are tensioned to 0.45 f_u instead of 0.68 f_u. By proportion, the approximate horizontal load capacity of these shear keys has been reduced to:

S1 $$42\text{x}\frac{12}{48}\text{x}\frac{0.45}{0.68} = 7\text{MN}$$ [Eq. 1]

S2 $$42\text{x}\frac{16}{48}\text{x}\frac{0.45}{0.68} = 9\text{MN}$$ [Eq. 2]

The anchor bolts at shear keys S3, S4 are from a separate lot from those at S1, S2 and have been tensioned to 0.7 f_u since April (communication from Peter Lee of MTC 2013 August 7.)

The “present” horizontal load capacity of the shear keys and bearings at Pier E2 can be summarized as follows:

Table 2:

	Longitudinal Direction	**Transverse Direction**
Shear Key S1	7 MN	7 MN
Shear Key S2	9 MN	9 MN
Shear Keys S3, S4	42 MN each (20 mm Gap)	42 MN each
Bearings B1, B2, B3, B4	15 MN each (20 mm Gap)	30 MN each (20 mm Gap)
Total Required Capacity	50 MN	120 MN
Total Capacity Supplied	16 MN	100 MN

The “present” transmission of horizontal seismic forces at Pier E2 is shown in Figure 2.

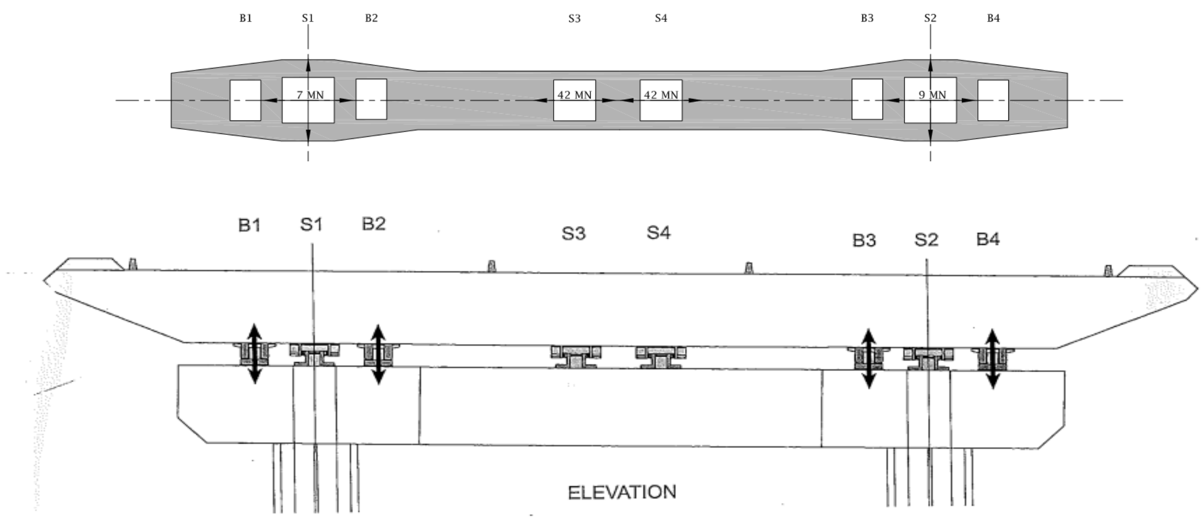

Figure 2: “Present” Transmission of Horizontal Seismic Forces at Pier E2

The “present” configuration of bearings and shear keys at site is unable to mobilize sufficient longitudinal and transverse resistance to handle the demands of the 1500 year ground motions.

4.3 Stage 3: Transmission of Horizontal Seismic Forces from the Bridge Superstructure into Pier E2 under "Proposed Temporary" Conditions

The "proposed temporary" conditions comprise shimming of bearings B1, B2, B3, B4 to eliminate their existing 20 mm longitudinal and transverse bearing travel gaps.

The transmission of horizontal seismic forces would then take place through the four bearings B1, B2, B3, B4 and the two shear keys S3, S4. The small contributions of shear keys S1, S2 during these proposed temporary conditions can be neglected.

The "proposed temporary" horizontal load capacity of the shear keys and bearings at Pier E2 can be summarized as follows:

Table 3:

	Longitudinal Direction	Transverse Direction
Shear Key S1	7 MN (Negligible)	7 MN (Negligible)
Shear Key S2	9 MN (Negligible)	9 MN (Negligible)
Shear Keys S3, S4	42 MN each (20 mm Gap)	42 MN each
Bearings B1, B2, B3, B4	15 MN each	30 MN each
Total Required Capacity	50 MN	120 MN
Total Capacity Supplied	60 MN	204 MN

The "proposed temporary" transmission of horizontal seismic forces at Pier E2 is shown in Figure 3.

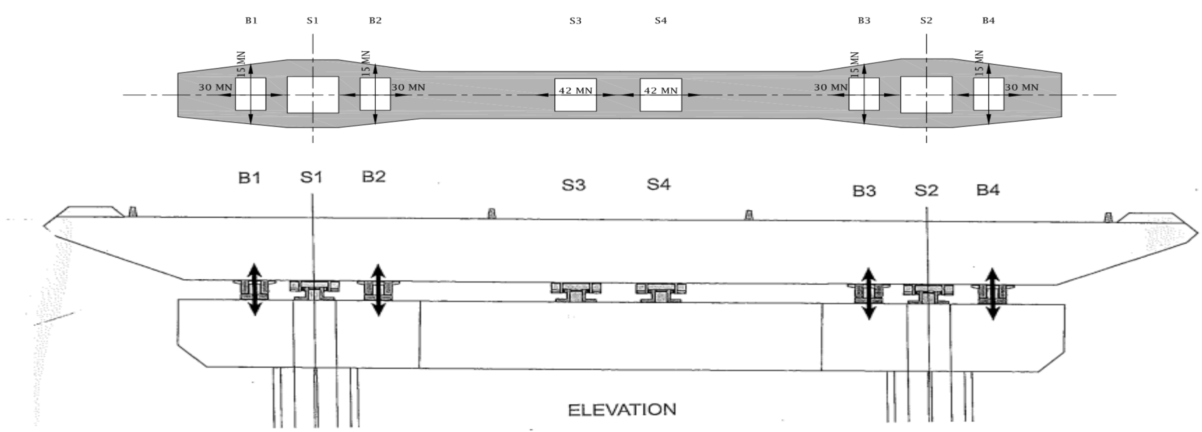

Figure 3: "Proposed Temporary" Transmission of Horizontal Seismic Forces at Pier E2

The “proposed temporary” combined horizontal load capacities of the bearings and shear keys at Pier E2 exceed the seismic demands in both the longitudinal and lateral directions.

The precise distribution of the horizontal forces in the transverse direction between the bearings and shear keys is indeterminate. However, the overall demand over capacity ratio in the transverse direction is only about 0.6, so that significant variations above the average demand on any one component can be tolerated.

Some small thermal forces may occur in the crossbeams of the pier and superstructure at Pier E2 due to differential temperatures between the superstructure and substructure.

4.4 Stage 4: Transmission of Horizontal Seismic Forces from the Bridge Superstructure into Pier E2 under “Revised Final Design” Conditions

The “revised final design” capacities and load paths replicate the “as originally designed” stage, except that the actions of the anchor bolts connecting the bases of shear keys S1 and S2 to the crossbeam of Pier E2 are replaced by a new array of post-tensioned cables anchored on the faces of the Pier E2 crossbeam and pulling downwards on steel yokes located over the shear key baseplates; see Figure 4.

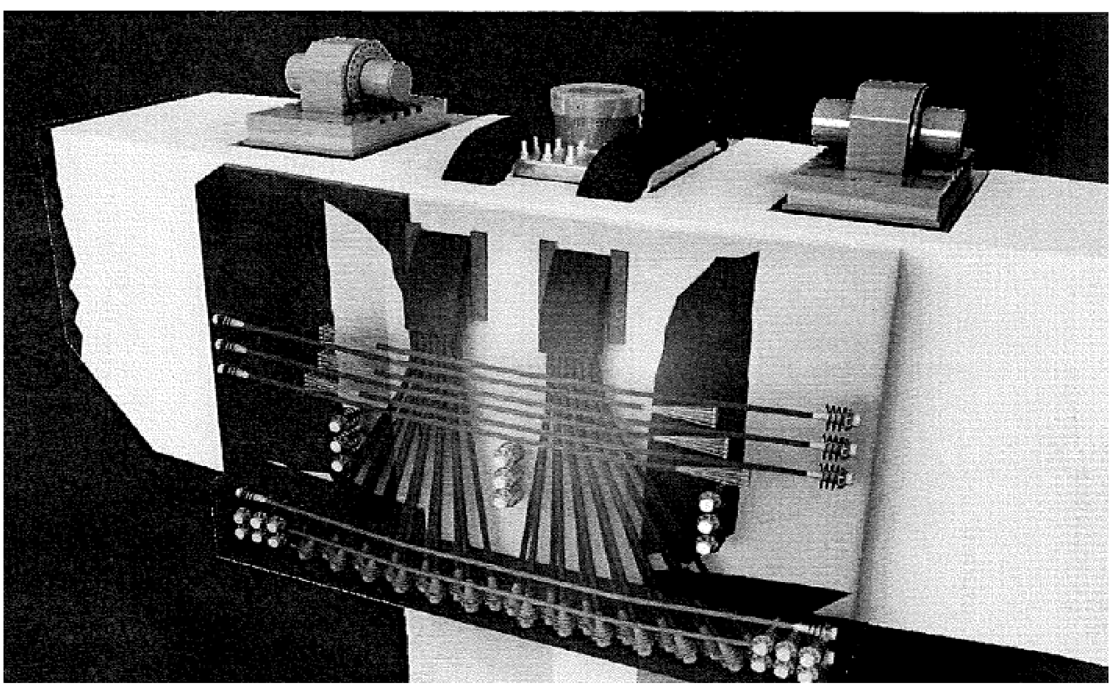

Figure 4: Steel Saddle Retrofit of Shear Keys S1, S2

Once these post-tensioned steel saddle retrofits have been completed at shear keys S1 and S2, then the shims will be removed from bearings B1, B2, B3 and B4.

The "revised final design" horizontal load capacity of the shear keys and bearings at Pier E2 can then be summarized as follows:

Table 4:

	Longitudinal Direction	**Transverse Direction**
Shear Keys S1 & S2	42 MN each	42 MN each
Shear Keys S3 & S4	42 MN each (20 mm Gap)	42 MN each
Bearings B1, B2, B3 & B4	15 MN each (20 mm Gap)	30 MN each (20 mm Gap)
Maximum Seismic Displacement	1.1 m	0.4 m
Total Required Capacity	50 MN	120 MN
Total Capacity Supplied	84 MN	168 MN

The "revised final design" transmission of horizontal seismic forces at Pier E2 in shown in Figure 5.

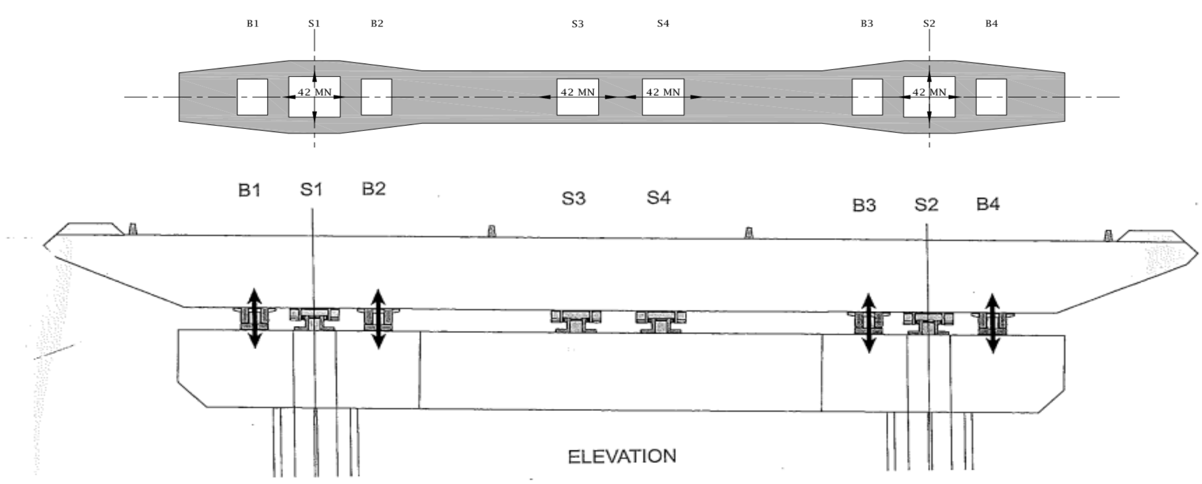

Figure 5: "Revised Final Design" Transmission of Horizontal Seismic Forces at Pier E2

5 Review of the Capacities of Critical Components in the "Proposed Temporary" Conditions

5.1 Shear Keys S3, S4

These components have the largest individual horizontal seismic design load capacity of 42 MN each and in combination with the shimmed bearings bring significant transverse overcapacity to the temporary fix.

During the temporary conditions Shear Keys S3, S4 are working together with Bearings B1, B2, B3, B4 to resist transverse seismic loads and the overall transverse demand is only 60% of the capacity, so the shear keys are in general working below their design capacity. The precise transverse load sharing between shear keys S3, S4 and bearings B1, B2, B3, B4 is indeterminate.

The S3, S4 shear key anchor bolts are a different lot from those which failed at shear keys S1 and S2 and have been tensioned to 0.7 f_u since 2013 April. Based on the findings of the Report on the anchor bolts[1], their short term vulnerability to stress corrosion cracking is low. The bolts are scheduled to be replaced after the bridge opening.

5.2 Bearings B1, B2, B3, B4

During temporary conditions the bearings and shear keys combined are working at 83% of design capacity in the longitudinal direction and 60% of design capacity in the transverse direction to resist 1500 years seismic design forces.

The Finite Element Analyses under full design loads presented by the designer[2] show low stresses in the bearings under longitudinal loads and only a few local stress concentrations under transverse loads.

The bearing hold-down analysis uses a coefficient of friction of 0.67 between steel and concrete based on the clamping force achieved with anchor bolts tensioned to 0.70 f_u. Localized yielding occurs at the edges of the bearing lower housing under maximum design transverse seismic loads. This localized yielding may be a result of computer modeling limitations. The amplitude and extent of the yielding is not of concern. As for the shear keys S3, S4, the total transverse load demand available under temporary conditions is only 60% of capacity.

5.3 Shims

Taking into account the local high steel-to-steel bearing stresses identified in the Finite Element Analyses of transverse loadings on the bearings, the shim plates may experience yield strains in a seismic event. The use of high yield strength shim material should be considered.

6 Conclusions

6.1 General Structural Conclusions

It is concluded that introducing shims into bearings B1, B2, B3, B4 at Pier E2 to inhibit their designed longitudinal and transverse bearing travels of 20 mm will create stiff load paths which will temporarily replace the adjacent stiff load paths for horizontal seismic loads through shear connectors S1 and S2, while the latter are rehabilitated.

The new load paths do not affect global seismic stiffnesses, seismic forces or displacement demands.

The new load paths do not affect the strength of Pier E2.

All design provisions to resist uplift at Pier E2 are preserved.

6.2 Specific Component Conclusions

During the temporary conditions, the transverse seismic loads of 120 MN from the superstructure are transmitted to Pier E2 through shear keys S3, S4 and bearings B1, B2, B3, B4 which together have a transverse design capacity of 204 MN. Thus global demands are significantly less than design capacities.

The corresponding proportion of the design capacity utilized in the longitudinal direction is 83% with all of the longitudinal resistance originating in bearings B1, B2, B3, B4.

The “shear key failed” Finite Element Analyses of the bearing components show only minor local zones of high stress under full transverse and longitudinal design load.

We conclude that the bearings and shear keys are well able to handle the 1500 year seismic demands with the bearings in their shimmed configuration during the “proposed temporary conditions”.

The anchor bolts at both the bases and the tops of the shear keys S3, S4 and bearings B1, B2, B3, B4 are of different lots from those that failed at shear keys S1, S2. They have sustained axial stresses of 0.7 f_u since 2013 April and can be considered, in the short term, not to be vulnerable to stress corrosion cracking. They are scheduled to be replaced in the medium term.

7 Recommendation

It is recommended that the shim concept be installed at the earliest opportunity because it restores “as designed” seismic capacity at Pier E2 and will provide appropriate interim seismic protection to users of the new bridge when it is opened.

8 References

[1] Report on the A354 Grade BD High Strength Steel Rods on the New East Span of the San Francisco-Oakland Bay Bridge – with Findings and Decisions; Toll Bridge Program Oversight Committee.

[2] San Francisco-Oakland Bay Bridge Self-Anchored Suspension Span – Seismic Evaluation of SAS at E2 Pier prior to completion of shear keys S1 & S2 July 15, 2013 T.Y. Lin – Moffatt & Nichol.

San Francisco-Oakland Bay Bridge Seismic Retrofit Project

Independent Review of Analysis and Strategy to Shim Bearings at Pier E2 to Achieve Seismic Design Requirements

Modjeski and Masters, Inc.

August 9, 2013

BAY AREA TOLL AUTHORITY

MODJESKI and MASTERS
Experience great bridges.

Independent Review of Analysis and Strategy to Shim Bearings at Pier E2 of the SFOBB to Achieve Seismic Design Requirements

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Executive Summary and Conclusions

Modjeski and Masters, Inc. (MM) was retained to conduct a peer review of the suitability of the "Shim Concept" to provide sufficient seismic capacity at Pier E2 of the San Francisco-Oakland Bay Bridge (SFOBB) to withstand the Safety Earthquake Event (SEE) forces until the shear keys are repaired. The zone of the SFOBB considered is the load transfer mechanism at Pier E2 from the superstructure bearing and shear key supports to the tops of the columns.

In the current context, a peer review involves determining if the work being reviewed meets the overall project safety requirements. It is not necessary to agree completely in a quantitative sense so long as the conclusion of the review is adequately supported.

Based on the engineering studies described herein, MM concludes that the concept of temporarily shimming the bearings at Pier E2 of the San Francisco-Oakland Bay Bridge as described in the July 15th information package entitled "Seismic Evaluation of SAS at E2 Bent Prior to Completion of Shear Keys S1land S2", and the proposed details, will provide more than sufficient capacity between the superstructure and the strut at Pier E2 to resist the design Safety Evaluation Earthquake. The service and seismic design forces in Pier E2 are not changed significantly by the redirection of seismic forces resulting from the shimming. Therefore the response of the concrete strut and columns are not expected to be different. Other than the possibility of scratching the paint system either when the shims are inserted or as the bridge rotates in service, there does not appear to be any reasonable possibility of damaging the bridge as a result of installing and removing the shims. Assuming that the rest of the structure has been properly designed, we conclude that the safety of the traveling public is improved by moving traffic on to the new bridge.

The exact distribution of forces in the bearings and shear keys is highly dependent on the planned and accidental tolerances (gaps) between various components. A precise quantification of the forces is, for practical purposes, unknowable. It is, however, possible to make estimates of the reasonable range of possibilities rather than relying on a single solution; MM’s assessment is based on that approach. Various assumptions have been used to place reliance on the set of shear keys and on the set of shimmed bearings, separately and in combination. Analysis of the resulting distribution of forces was the basis for concluding that the temporary shimmed condition provides sufficient resistance to meet the demands from the SEE.

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Independent Review of Analysis and Strategy to Shim Bearings at Pier E2 of the SFOBB to Achieve Seismic Design Requirements

1. Authorization

Engineering services required to develop this report were authorized by the Bay Area Toll Authority by Agreement of July 16th, 2013.

2. Scope

The following technical Scope is excerpted from the Agreement of July 16th:

The services to be performed by Consultant shall consist of services requested by the Project Manager or a designated representative including, but not limited to, the following:

Task 1:

Consultant shall provide an independent review of the engineering analysis and strategy as related to the proposal to shim the existing bearings at Pier E2 to achieve the seismic design requirements of the new east span of the San Francisco-Oakland Bay Bridge.

Consultant shall conduct a peer review of the suitability of the "Shim Concept" to provide sufficient seismic capacity at Pier E2 of the San Francisco-Oakland Bay Bridge to withstand the Safety Earthquake Event (SEE) forces until the shear keys are repaired. In performing the work, Consultant shall use:

1. *Load demands at Pier E2 from the analysis of the SEE pushover by the engineer of record (EOR) as summarized in "Seismic Evaluation of SAS at E2 Bent Prior to Completion of Shear Keys S l and S 2" dated July 15, 2013* (July 15th information package)

2. *Details of the bearings, shear keys and cap beam at Pier E2 and the supports of shear keys and bearings in the orthotropic box girders provided by EOR; and*

3. *Other calculations, plan sheets, shop drawings and engineering summaries prepared or supplied by request to the EOR as may be needed.*

The zone of the SFOBB to be considered is the load at Pier E2 from the box girder bearing and shear key supports to the tops of the columns.

The following is expressly not included:

1. *Review of the failures of the 2008 anchor bolts;*
2. *Independent seismic analysis or evaluation other than force transfer in the zone identified above;*
3. *Any capacity of the box girders or the substructure other than the force transfer zone*

identified above;

4. *Any evaluation of the repair of the shear keys (S l and S2) or the resulting suitability;*
5. *Any evaluation of the original design of the cap beam at Pier E2 other than as may be needed to evaluate the changes to the load path caused by the temporary shimming; and*
6. *Any other function of the shear keys and bearings other than transferring seismic demands in the temporary shimmed condition.*

Deliverable:

Independent Review Report

3. Background

Figure 1 through Figure 9, from the July 15th information package, were provided by the EOR, TYLin International.

3.1 Seismic Forces and Assumed Behavior of Bearings and Shear Keys

The original load path with clearances preventing the bearings from participating in resisting horizontal and transverse shear is illustrated in Figure 1. The four bearings were intended to carry only vertical loads, the four shear keys were intended to carry transverse shear, and shear keys 1 and 2 were intended to carry longitudinal shear.

Shimming is proposed to close the gaps and provide a contact load path through the bearing to transmit longitudinal and transverse shear. The location of the shims is shown in Figure 2. Due to failure of many of the anchor bolts at shear keys S1 and S2, it has been decided that all shear capacity at these shear keys should be ignored until they are repaired. With the shims in place, the proposed load path is shown in Figure 3. The bearings carry vertical loads as in the original load path, but are now intended to carry the entire longitudinal shear and a portion of the transverse shear. Shear keys 3 and 4 are still intended to carry some of the transverse shear, but the distribution of shear between the bearings and shear keys requires consideration. With shear keys S1 and S2 considered ineffective due to anchor bolt fractures, the longitudinal shimming is required in any event to provide an effective load path in that direction until the permanent repair of shear keys S1 and S2 in completed.

The Safety Evaluation Earthquake (SEE) demands and the nominal shear capacities of the bearings and shear keys for three scenarios (load paths) are shown in Figure 4; load path "A" is as designed, load path "C" is based on the shimming of the bearings and the participation of shear keys 3 and 4, i.e. the starting point for this investigation.

The magnitudes and directions of seismic forces given in the July 15th information package, excerpts of which are repeated below, are accepted without review per scope:

Bearing forces were extracted from a seismic (time history) analysis of the self-anchored suspension bridge including the bearings and shear keys. The total longitudinal, transverse, and vertical loads transferred from the westbound and eastbound box girders to Pier E2 were extracted from the analysis and distributed to the bearings and shear keys in accordance with the plans. The bearing loads are shown in Table 1.

Normal functioning of the bearing corresponds to the case "Shear Key Engaged". The bearing is only required to carry vertical loads. These are either downwards - case C - or upwards - case U. Upwards loads are of greatest concern and are addressed in this report. A "safety factor" of 1.4 is applied to the calculated loads from the seismic analysis.

The bearing is also intended to function as a secondary mechanism to resist longitudinal and transverse loads should the shear keys fail. The three cases of greatest interest are those corresponding to the peak uplift on the bearing (case U), the peak transverse load (case T), and the peak longitudinal load (case L). In each case the orthogonal loads occurring simultaneously with the peak loads are also tabulated (and analyzed). These loads are applied with a "safety factor" of 1.0, since they are based on the conservative assumption that the shear key has failed.

Bearing Forces (SF = 1.4)

Case	*Case*	*Trans.*	*Long.*	*Vert.*
Shear Key Engaged	*C*	*0*	*310*	*81104*
(Load Path A)	***U***	***0***	***108***	***-13355***

Bearing Forces (SF = 1.0)

Case	*Case*	*Trans.*	*Long.*	*Vert.*
	C	*10799*	*4770*	*57932*
Shear Key Failed	***U***	***25287***	***1628***	***-9539***
(Load Path B&C - See	***T***	***30496***	***8186***	***16441***
Note)	***L***	***1340***	***13232***	***19255***

Note: The same seismic demands are conservatively assumed for Load Path C.

Table 1, Bearing Loads

As mentioned previously, the loading on the model is assigned at the CG of the bearing shaft, which transfers the focus from the bearing upper housing to the bearing lower housing.

The load is modeled as pressure loading applied at relevant surfaces, with some simplifications.

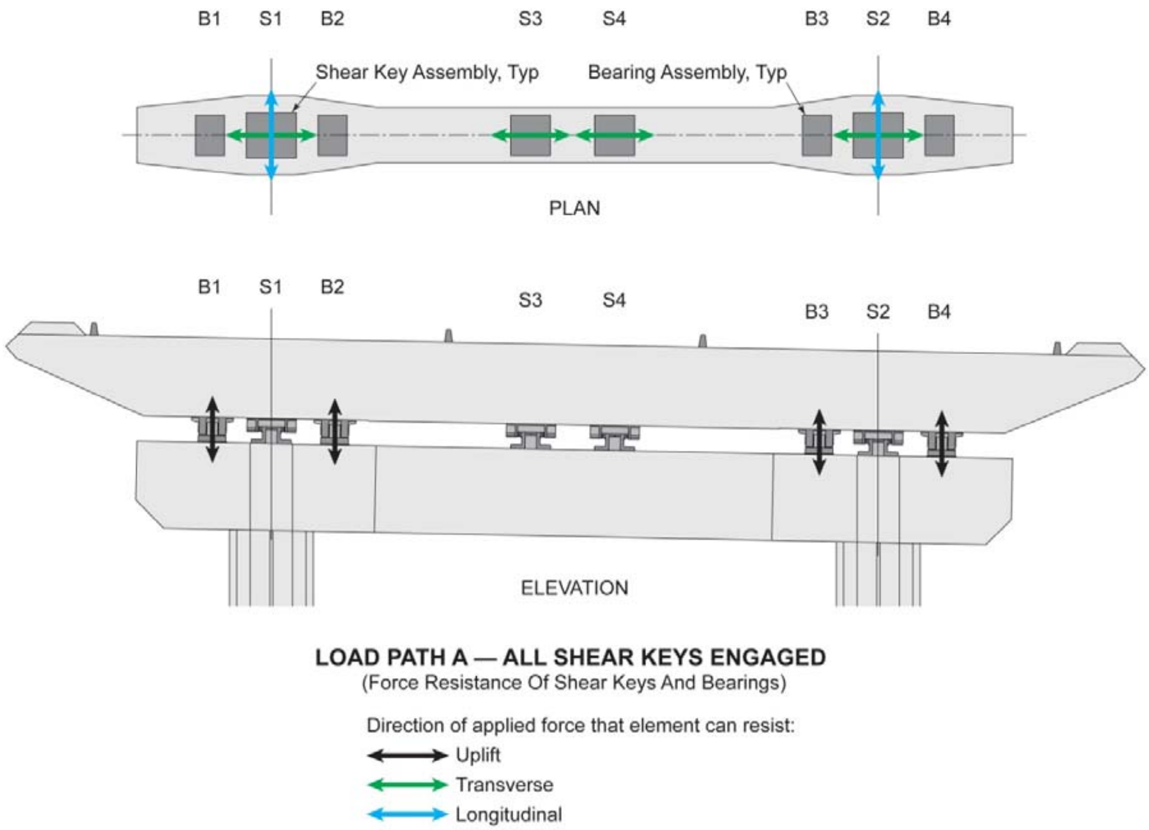


Figure 1 - Original load path through shear keys and bearings

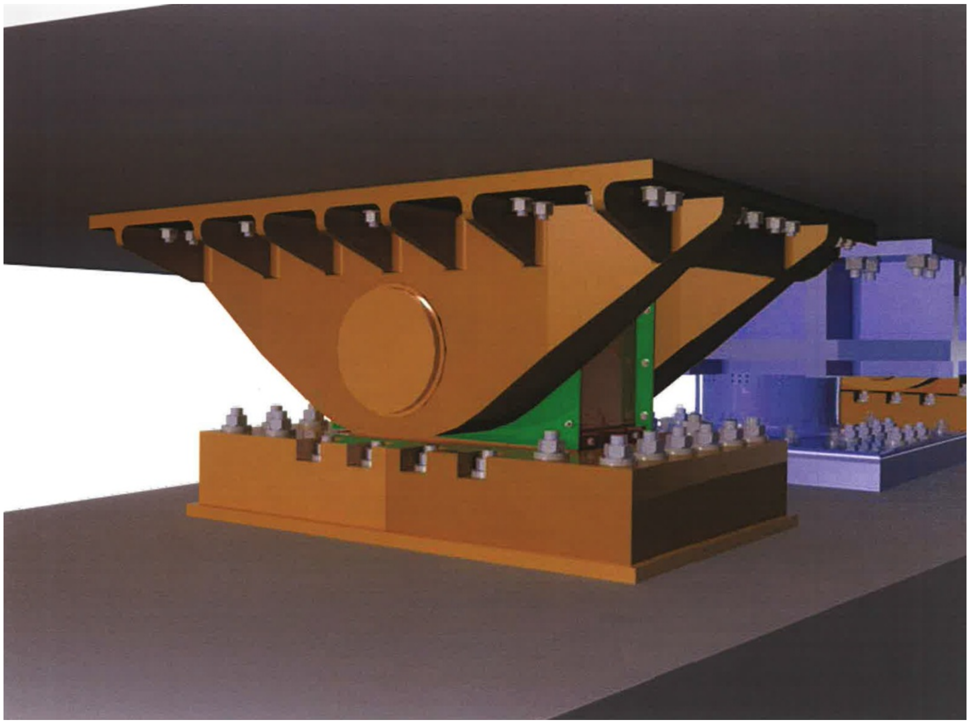


Figure 2 - Shimmed bearing

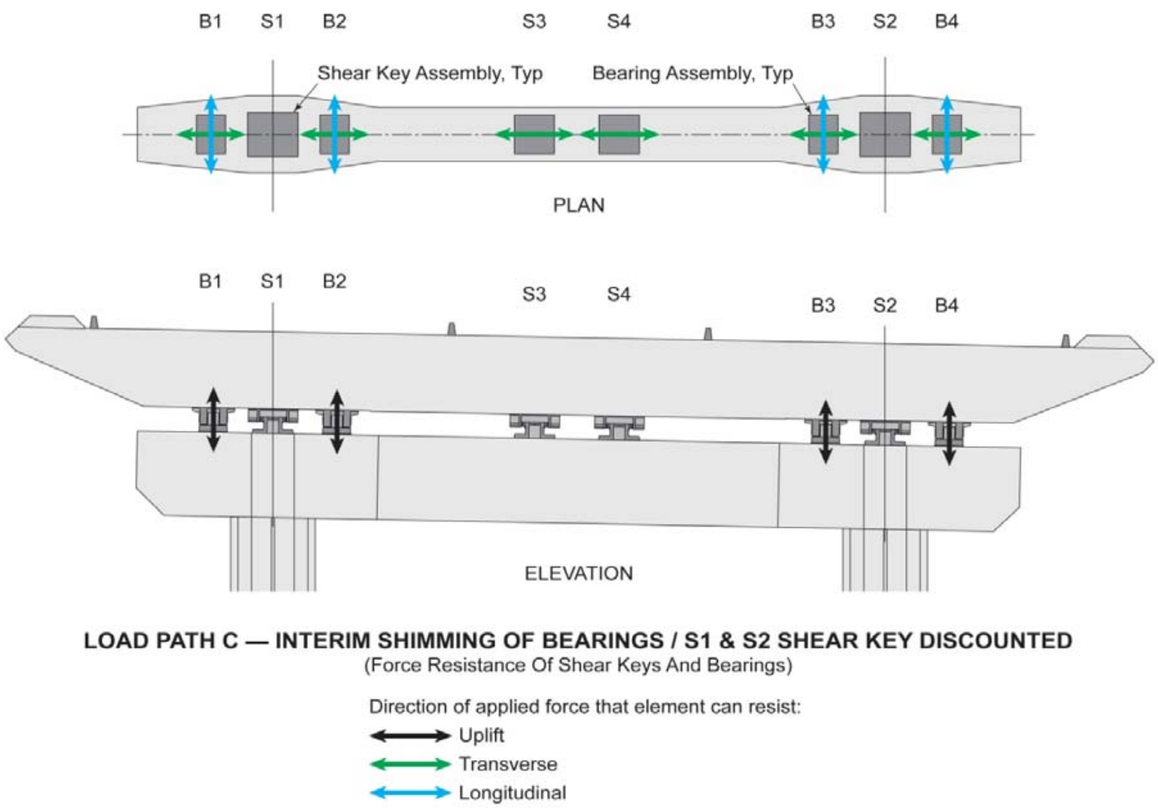
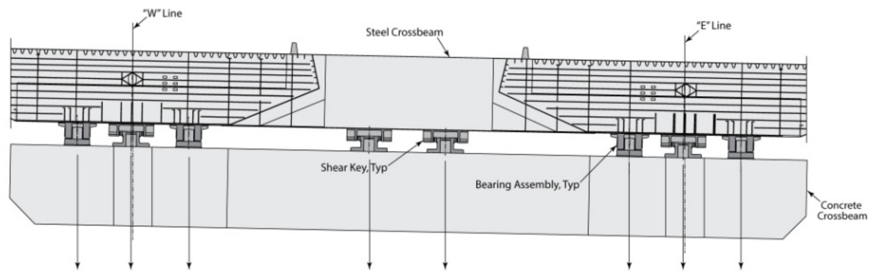


Figure 3 - Revised load path through shear keys and bearings



		Direction	B1	S1	B2	S3	S4	B3	S2	B4	TOTAL (MN)	SAFETY FACTOR
DEMAND	SEISMIC DEMAND FOR SEISMIC SAFETY EVALUATION (SEE) EARHQUAKE	Long									50	
		Trans									120	
CAPACITY	LOAD PATH A: (ALL SHEAR KEYS ENGAGED)	Long	0[(a)]	42	0[(a)]	0[(c)]	0[(c)]	0[(a)]	42	0[(a)]	84	168%
		Trans	0[(b)]	42	0[(b)]	42	42	0[(b)]	42	0[(b)]	168	140%
	LOAD PATH B: (ALL SHEAR KEYS DISCOUNTED) (BEARINGS ENGANGED)	Long	15	0	15	0	0	15	0	15	60	120%
		Trans	30	0	30	0	0	30	0	30	120	100%
	LOAD PATH C: (INTERIM SHIM OF BEARINGS) (S1 & S2 SHEAR KEYS DISCOUNTED)	Long	15	0	15	0[(c)]	0[(c)]	15	0	15	60	120%
		Trans	30	0	30	42	42	30	0	30	204	170%

a. 30 mm gap in the longitudinal direction. Bearing (B1, B2, B3, and B4) engage after 30 mm gap is closed by displacement.
b. 20 mm gap in the transverse direction. Bearing (B1, B2, B3, and B4) engage after 20 mm gap is closed by displacement.
c. 43 mm gap filled with neoprene open cell. Shear Keys (S3 and S4) engage in the longitudinal direction after 43 mm gap is closed by displacement.

Figure 4 - Reported demand and capacities of shear keys and bearings

3.2 Material Properties (Provided by EOR, anchor bolt capacity confirmed by MM)

- Concrete strength: f'_c = 55.2 MPa (8 ksi)
- Reinforcing steel: F_y = 414 MPa (60 ksi)
- Structural steel plates: F_y = 354 MPa (50 ksi)
- Steel castings: (354 MPa, 550 MPa (50 ksi, 80 ksi)
- Anchor rods were greased and taped:
  - Tensile area = 5.97 sq in
  - Force @ F_u=140 ksi for 3" bolt = 835.8K = 3.718 MN tensile strength
  - Stressed to 0.75: tensile force = 2.789 MN per bolt
  - Assuming 10% loss: 2.510 MN per bolt

- Prestressing strand: F_u = 1,862 MPa (270 ksi)

3.3 Previously Stated Shear Capacities of Bearings and Shear Keys

The bearings have split base plates such that the longitudinal capacity is one-half of the transverse capacity. The steel-steel coefficient of friction was assumed to be 0.50 yielding a sliding capacity 1.25 MN per bolt. This extends to 15.0 MN for 12 bolts in the longitudinal direction and 30.0 MN for 24 bolts in the transverse direction. (See possible reduction due to load path through hold down assembly discussed below.)

For the shear keys, the nominal capacity provided by the EOR was 42 MN based on first yielding indicated by a finite element analysis. For shear keys 3 and 4, the sliding capacity was determined to be 81 MN using a steel-concrete coefficient of friction of 0.67 and 48 bolts. This is much greater than the first yield capacity.

4. Approach

4.1 General

As indicated in the Scope, this investigation is focused on the force transfer zone between the bottom of the orthotropic box girders (OBGs) and the Pier E2 cross girder (CG) to the interface of the strut and columns. All structural issues related to the change in load path resulting from shimming the bearings and discounting shear keys 1 and 2 are believed to be concentrated in this area which is comprised of the pier strut including the contact surface and load transfer between the base plates and the concrete, the bearings and shear keys, and the capacity of the OBGs and cross girder to transfer loads to the bearings and shear keys.

The following areas were investigated relative to the revised loads resulting from the shimmed bearing concept:

1-Analysis of load transfer in bearings and shear keys including the capacity of friction interfaces, a first level approximate analysis of changes in clamping forces in bearings, and a more refined analysis of the effects of gaps and nonlinearities on load sharing among shear keys and bearings.

2-Capacity of OBGs and the Pier E2 cross girder to deliver load to the shimmed bearings and shear keys, respectively.

3-Capacity of the pier strut.

4-Transfer of horizontal load from bearing base plate to strut to column.

5-Consideration of potential to damage the permanent bearings or other components if shims are installed.

The exact distribution of forces in the bearings and shear keys is highly dependent on the planned and accidental tolerances (gaps) between various components and is, for practical purposes, unknowable. However, it is possible to make estimates of the reasonable range of possibilities rather than placing reliance on a single solution; MM's assessment is based on that approach, sometimes referred to as bounding the solution. Various assumptions have been used to place reliance on the set of shear keys and on the set of shimmed bearings, separately and in combination. At the extreme ends of the bounds, it could be assumed that the shear keys alone carry the transverse component of the SEE, or that the bearings alone carry the load. Other acceptable joint participation possibilities within these bounds may also be found to provide sufficient capacity. The possibility of unequal distribution among the individual shear keys and individual bearings in the two extremes should be considered at least subjectively.

The difference between the individual forces provided in the longitudinal and transverse directions and a vector sum of those forces was found to be small, on the order of a few percent; the individual forces will often be used herein for simplicity.

4.2 Analysis of Load Transfer in Bearings and Shear Keys

4.2.1 Free Body Diagrams

Figure 5 through Figure 9 illustrate the basic flow of forces through the shear keys and the bearings. For both types of assemblies, the shear from the seismic event is transferred from the upper housing to a lower housing at discrete points, shown as the shear key bushing or the shaft in the case of the bearing. The rest of the load paths in Figure 5 through Figure 9 are global statics and will be used as a starting point for the investigations reported herein. The distribution of stress within the assemblies and the participation of individual anchor bolts can be more complex.

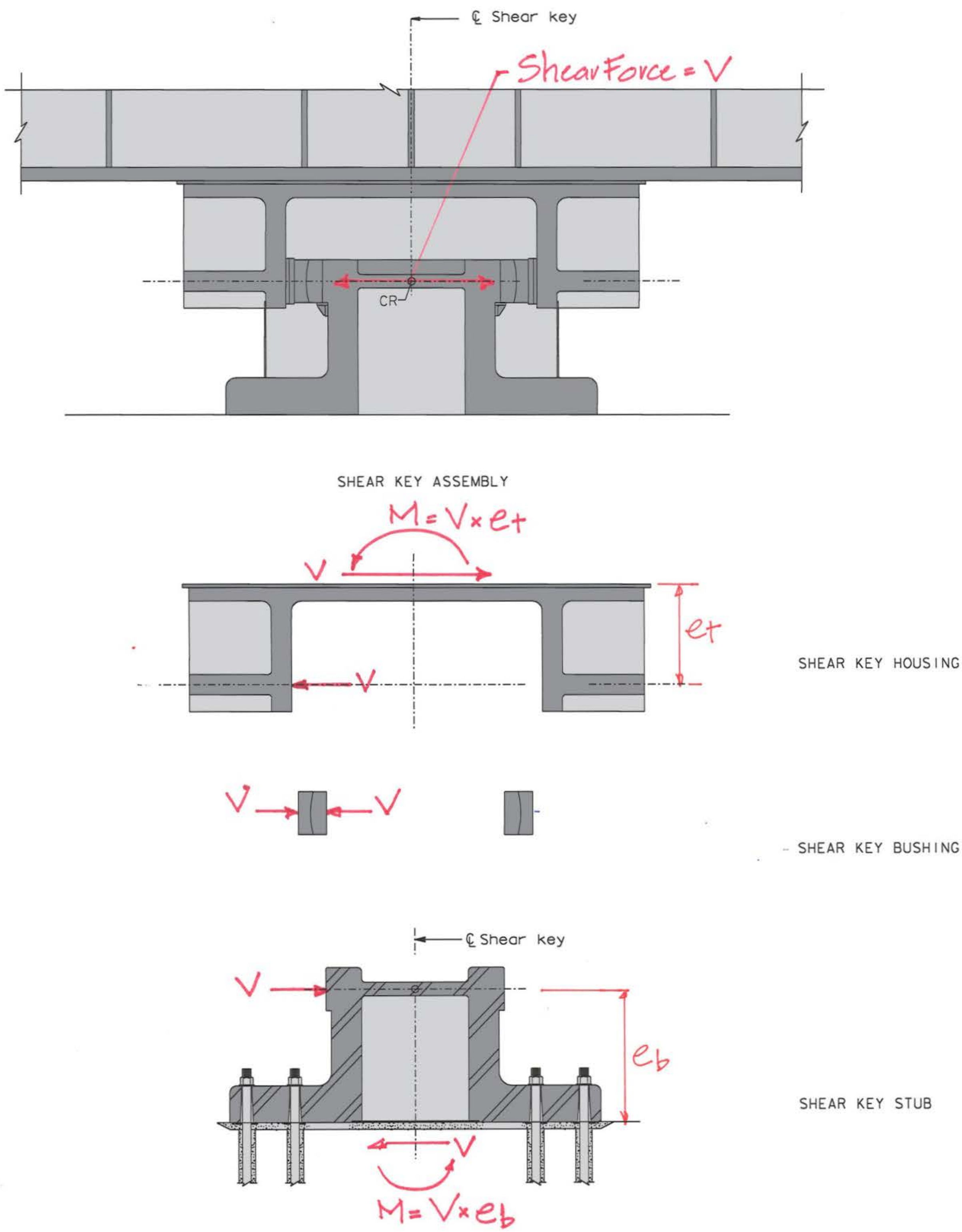


Figure 5 - Load path through shear key

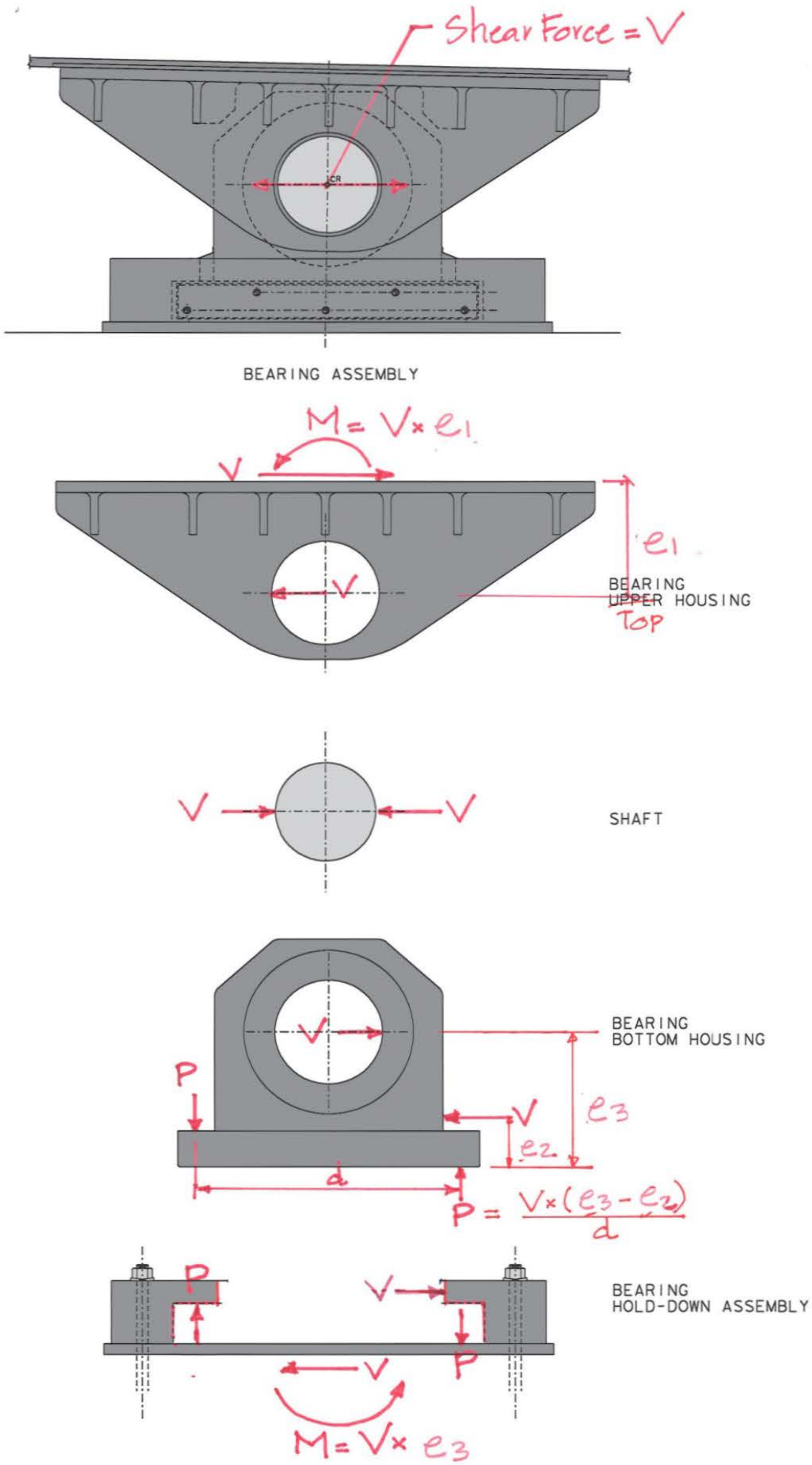


Figure 6 - Load path through bearing - longitudinal shear

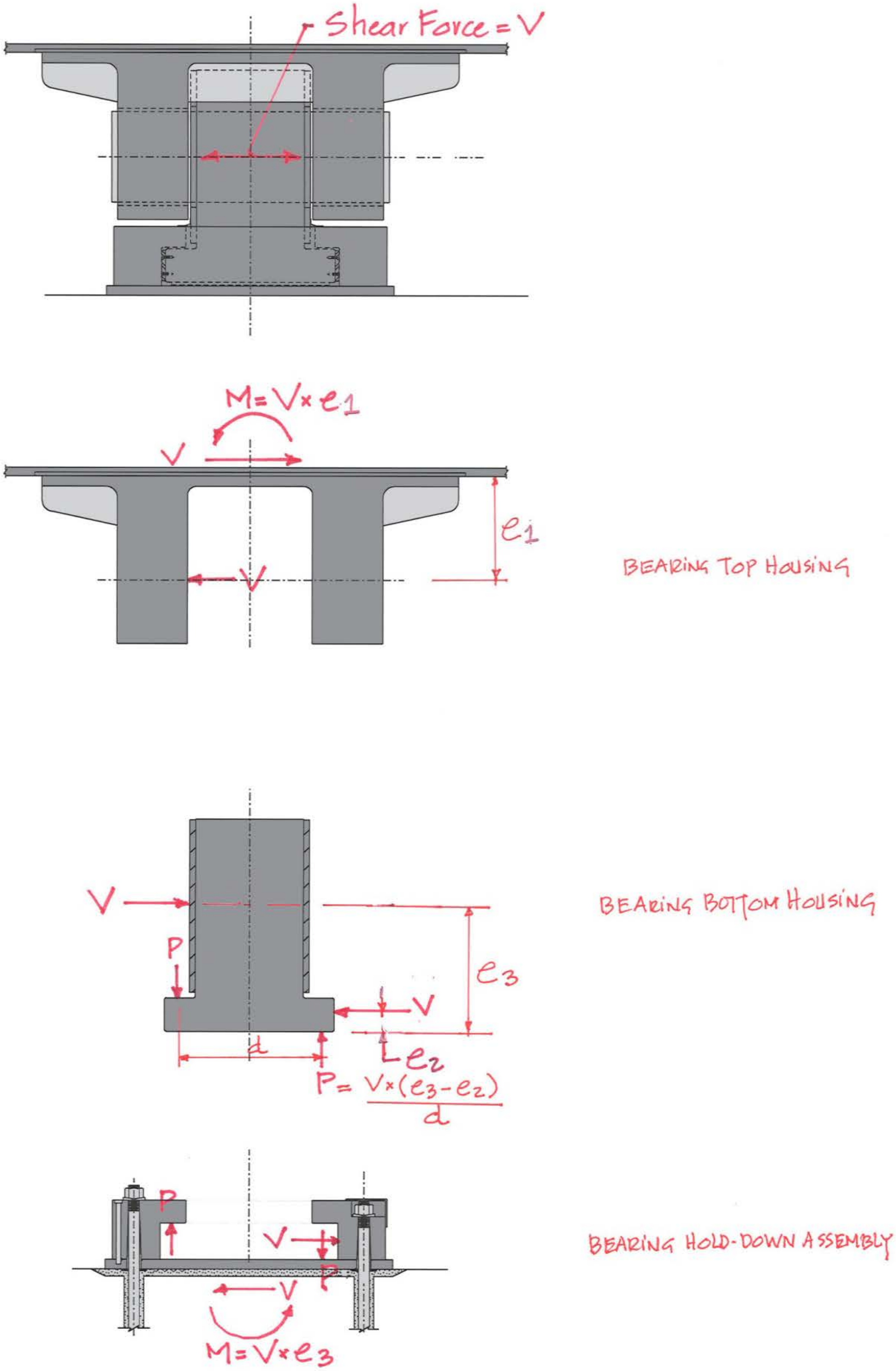


Figure 7 - Load path through bearing - transverse shear

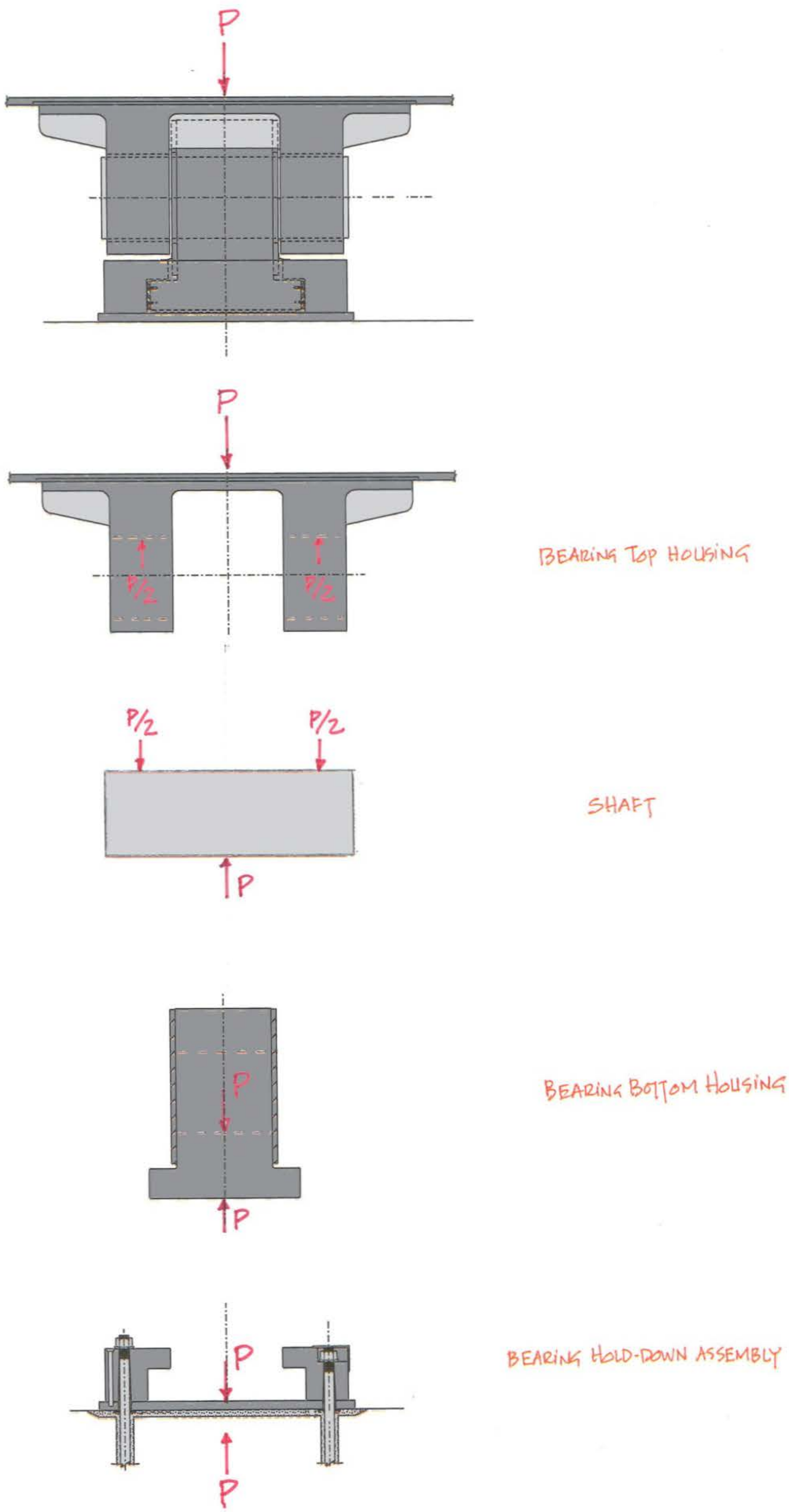


Figure 8 - Load path through bearing - compression

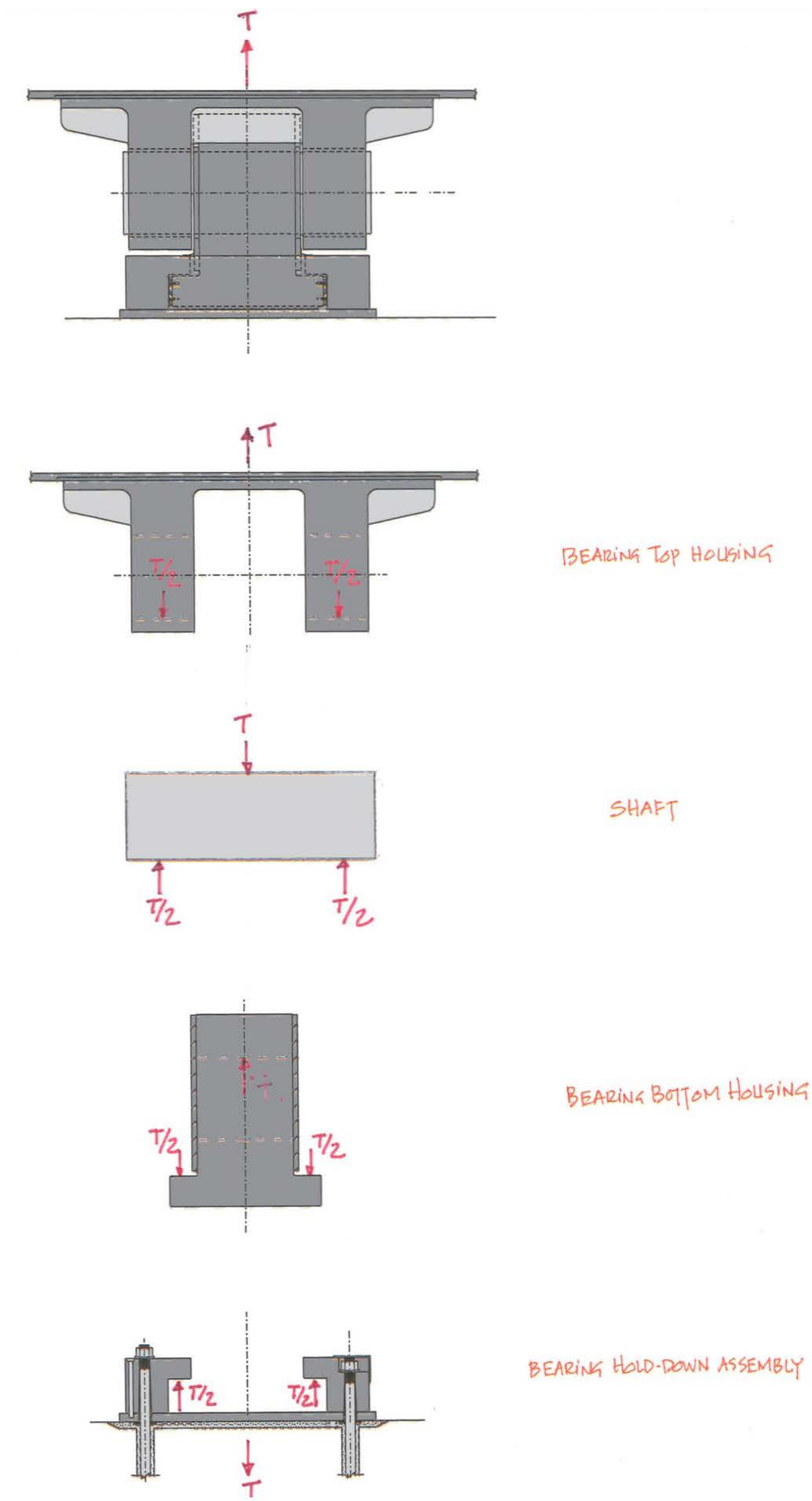


Figure 9 - Load path through bearing - uplift

4.2.2 Coefficients of Friction at Interfaces

The coefficient of friction of 0.50 used for the steel-steel interface in the bearings is consistent with the AASHTO Class B coating factor for slip-critical high strength bolts. EOR has confirmed orally that Class B surface preparation was required; written confirmation in specifications or shop drawings has been requested**.** Since the steel-steel coefficient of friction is less than the steel-concrete coefficient there is no need to check the latter capacity.

The coefficient of 0.67 used for steel anchored to concrete in the shear keys is less than (more conservative than) 0.70 used by AASHTO for shear friction. The bottoms of the base plates were machined (roughened) with 5 mm deep recesses as shown on shop drawings provided by the EOR further improving the performance. Friction capacity using the 0.67 factor results in about 81 MN or about twice the nominal capacity of 42 MN provided in the July 15th information package. The corresponding capacity of the steel to steel interface at the upper housing is approximately 100 MN.

4.2.3 Capacity of Bearings and Shear Keys (individual and combined) at 100% SEE – Approximate Mechanics Analysis

First yield capacity for the shear key stub is about 76 MN for bending alone and 73 MN in shear alone based on simple mechanics and a yield strength of 345 MPa (50 ksi). Conservatively using the maximum shear and bending stresses as though they occurred at the same point on the stub, results in a von Mises capacity of 53.3 MN at first yield. MM's finite element analysis of the shear key stubs indicates first yield at 49 MN. The EOR reports that first yield load of the stub based on FEA is 42 MN and that value will be used in this section.

Referring to the bottom panel of Figure 7, the load path in the bearings causes compression due to overturning from the transverse shear to go directly into base plate through a lubricated surface so as to bypass the hold down assembly while the tensile component goes directly into the tang on the hold down assembly resulting in a net reduction of clamping force on the steel-steel interface. As shown in the bottom panel of Figure 9, uplift goes into the hold down tang which also reduces clamping although this is partially offset on one side by overturning compression. (The overturning effect is much larger than the axial effect). The net result is a reduction in clamping force and an associated reduction in shear capacity compared to that indicated by the estimates in the July 15th information package (bearings: 30 MN transverse and 15 MN longitudinal). Results are summarized below and more details are given in Appendix 1.

For the load case with the maximum transverse force, the shear capacity of the bearings is currently estimated, based on rough hand calculations, to be about 25.7 MN or about 86% of the nominal 30 MN capacity. The capacity of the 4 bearings alone would be 103 MN, less than the SEE demand of 120 MN, so some participation from the shear keys is needed to resist the SEE. Using the concept of bounding the possible solutions, consider that all of the shear keys and bearings engage equally and are limited to 86% of their nominal total shear capacity (42 MN and 30 MN, respectively) to protect the bearings. The transverse resistance would be about 175MN, greater than the 120 MN needed for the maximum transverse force case. Alternatively, if the shear keys reached their nominal 42 MN capacity and the bearings were limited to their shear capacity based on the reduced clamping force (25.7 MN), the total resistance would be 187 MN.

In the maximum uplift case, the shear capacity of the bearings is further reduced to 21.7 MN or 72% of their capacity based on the unreduced clamping force for a total capacity of the 4 bearings of about 87 MN which is less than the transverse shear associated with the maximum uplift, estimated by proportion to be about 100 MN . Again, some participation from the shear keys is needed. Following the logic used above, if the total shear resistance was based on all bearings and shear keys being limited to 70% of their nominal capacity to protect the bearings, the resistance would be about 143 MN. This is greater than the 120 MN maximum transverse shear and even greater than the proportioned 100 MN transverse shear for the uplift case. If the shear keys reached their full resistance and the bearings reached their reduced capacity (21.7 MN), the shear resistance would be 171 MN.

As shown in the Section 4.2.4, in which the displacement compatible system behavior of the shear keys and bearings is discussed, MM's analysis shows that, for a range of assumed gaps, more of the transverse load is carried by the shear keys and less by the bearings than consideration of their nominal capacities would indicate. The net effect is that the displacement compatible demands on the bearings are smaller than the clamping reduced capacities presented in this section.

Further bounding the analyses, if full reliance were placed equally on the two shear keys each would have to carry 60 MN. This is greater than the nominal 42 MN capacity but less than their sliding capacity of about 81 MN. This implies that either some yielding of the shear key stub or housing, or participation of the bearings, would be necessary to carry the full load. This is investigated using more refined analysis in the next section.

Longitudinal shear in bearings is not affected by the reduced clamping force issue discussed above because the reduced clamping would be on the side of the split base plate which is neglected in calculating the shear capacity. Thus the 15 MN resistance per bearing or 60 MN total for 4 bearings given in Figure 4 is deemed appropriate and more than the 50 MN reported to be required by the SEE.

The shear keys are not affected by the reduced clamping force issue because tension and compression, from overturning, are both carried by the same elements so there is no net change in the clamping force.

4.2.4 Capacity of Bearings and Shear Keys (individual and combined) at 100% SEE – Refined Finite Element Analysis

A potential issue that was identified early in the review is the sharing of load between the shear keys and the bearings. Both the shear keys and the bearings, when fully engaged, provide a very stiff load path for the seismic forces between the OBG and Pier E2. As noted earlier, the bearings were designed with sufficient clearances to prevent them from transferring horizontal load. Some, but not all of these clearances will be addressed by the proposed shimming operations. In the transverse direction, there is a horizontal gap with an as-designed value of 2 mm, between the lower housing and the hold down assembly, which will remain after shimming. See Figure 10. Additionally, there is an as-designed 3mm vertical gap that allows the lower housing to rotate a small amount. These gaps allow a certain amount of free movement of the bearings before they become engaged in transferring lateral forces which will affect the distribution of loads between the shear keys and the bearings.

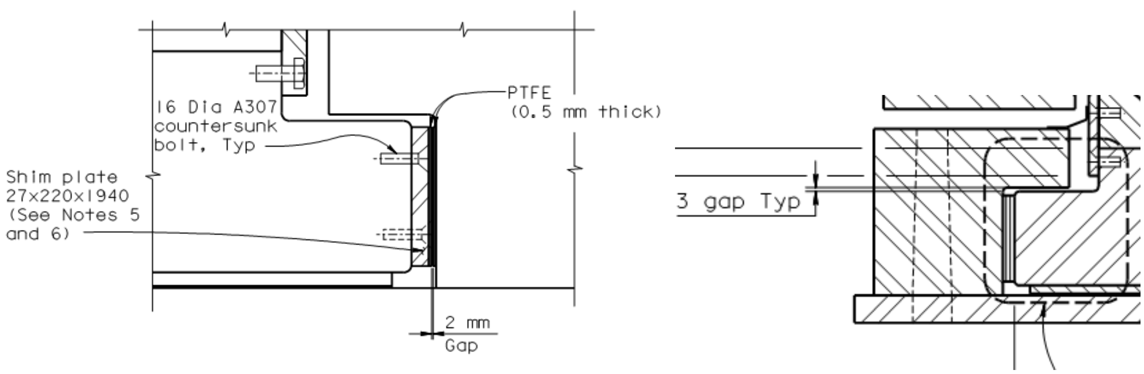


Figure 10 - Details from the design drawings showing 2 mm and 3 mm gaps

Two analyses were performed to evaluate the distribution of load between the shear keys and the bearings. A 3D FEA including the hold down assembly and the lower housing was performed to determine the force-displacement behavior in the as-designed condition. The model included the 2mm and 3mm gaps, as well as compression-only contact surfaces. Figure 11 shows a general view of the model and Figure 12 shows the deformed shape with stress contours. A separate FEA was performed on the top housing and its stiffness was included to find the total bearing force-displacement relationship. Additionally, the force-displacement relationship was calculated by hand using typical engineering approximations.

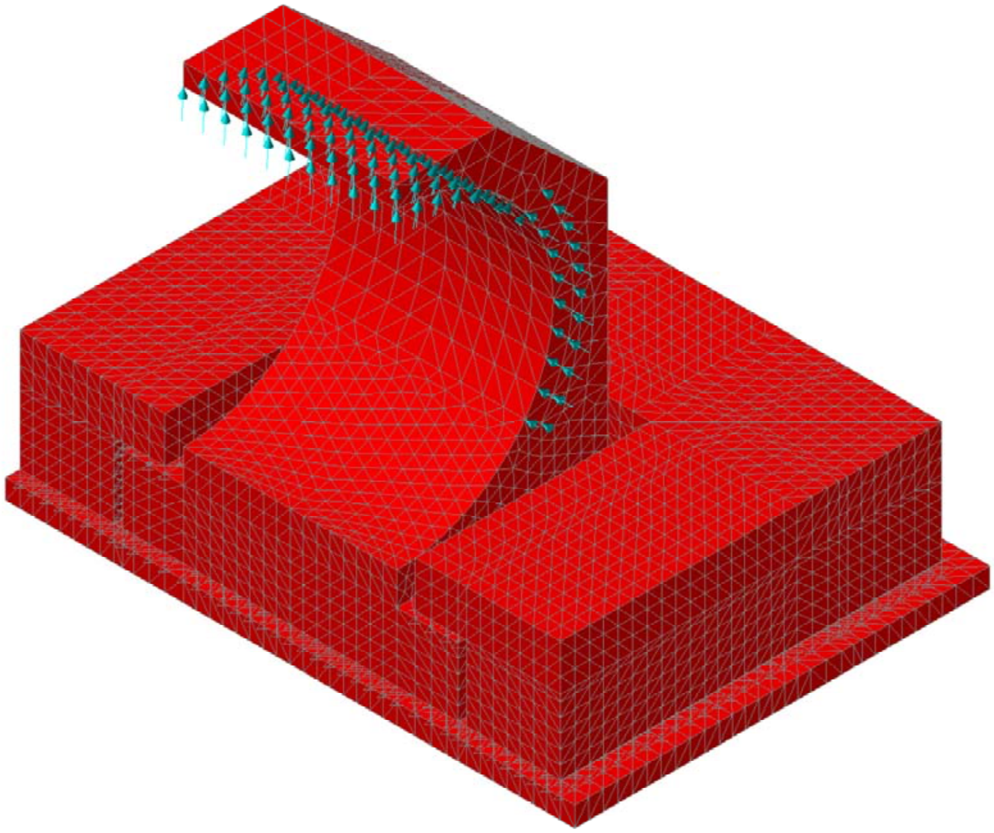


Figure 11 - Finite element model of the lower housing and hold down assembly

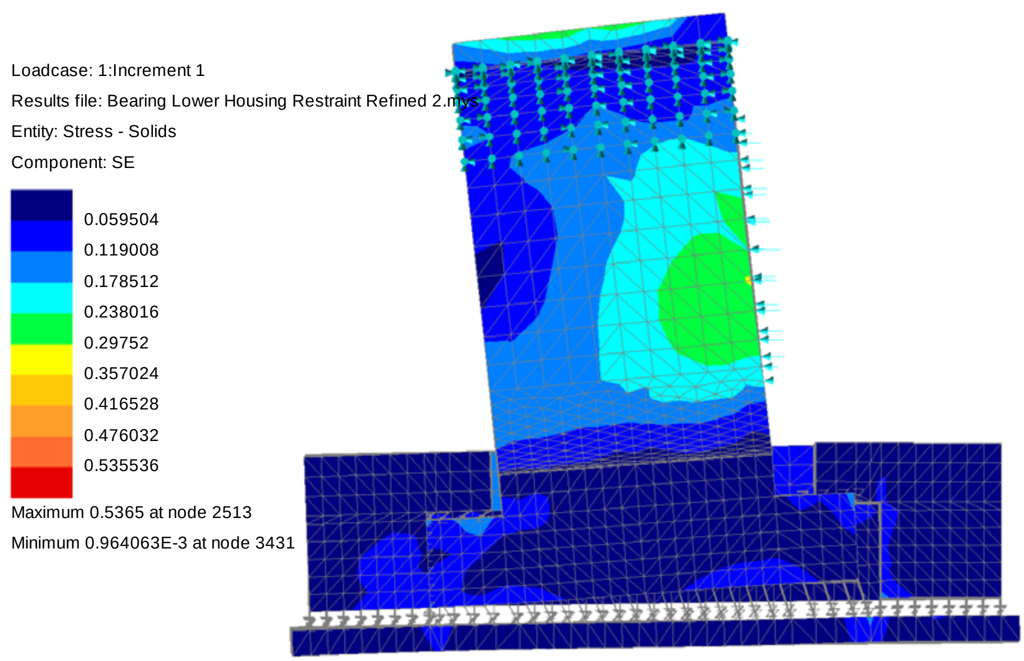


Figure 12 - End-on view of the deformed shaped under lateral and uplift loading

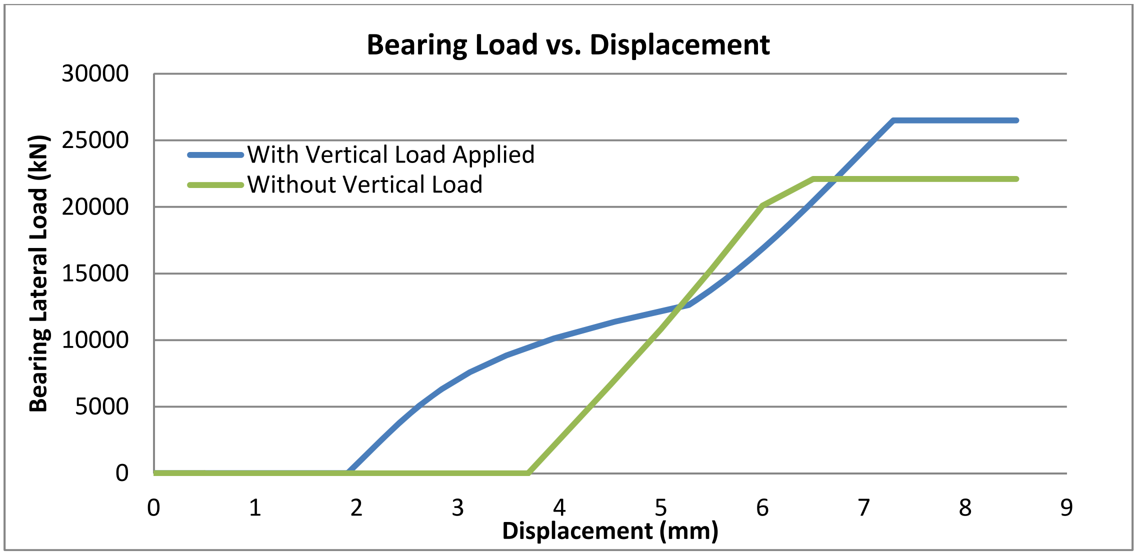


Figure 13 - Load vs. displacement for bearings with varying vertical load

Figure 13 shows the lateral load vs. displacement relationship for two conditions; with vertical load (uplift in the case shown) and with no vertical loads. There is a coupling between lateral displacement and vertical load through the rotation of the lower housing. Because the presence of vertical load during the times of large lateral loading cannot be guaranteed, the curve without vertical load was used in further evaluations. The plateaus in the graphs are the approximate load levels at which friction is overcome, and the bearings begin to slide on the steel-to-steel interface.

Finite element models were also developed for the shear key; both the stub and the housing were modeled in separate analyses. Because of the potential for exceeding the stated capacity of 42 MN for the shear keys, nonlinear material behavior was included in the models. The lateral stiffness of the shear keys was determined from these analyses, as well as from independent hand calculations. Figure 14 shows the load vs. displacement relationship for the shear key. The curve is quite linear to loads well in excess of the nominal capacity, with softening becoming apparent only at very large load levels (>70 MN). A 0.5 mm gap was assumed between the stub and the spherical ring. It was indicated that the actual free play in the shear keys may be larger and this is explored below.

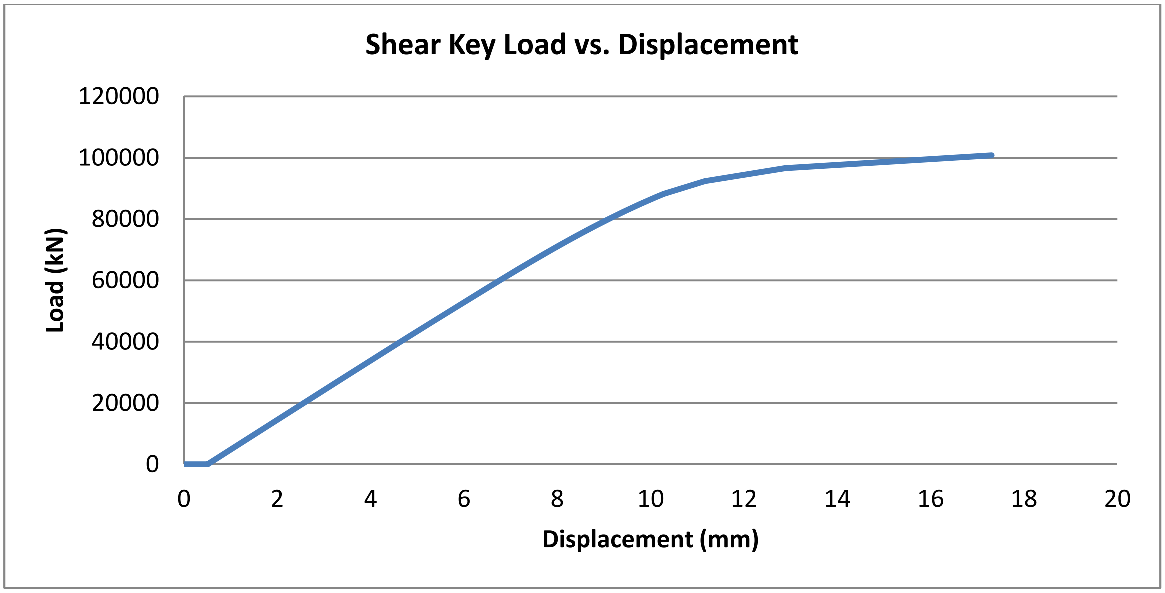


Figure 14 - Shear key load vs. displacement

A total force vs. displacement curve can now be assembled from the individual curves for the interface between the OBG and Pier E2 with 4 bearings and 2 shear keys. This is shown in Figure 15. Table 2 lists the forces in the shear keys and bearings for several displacements based on the curves above. Because the bearings do not carry load until the various gaps have closed, the shear keys carry a majority of the seismic load.

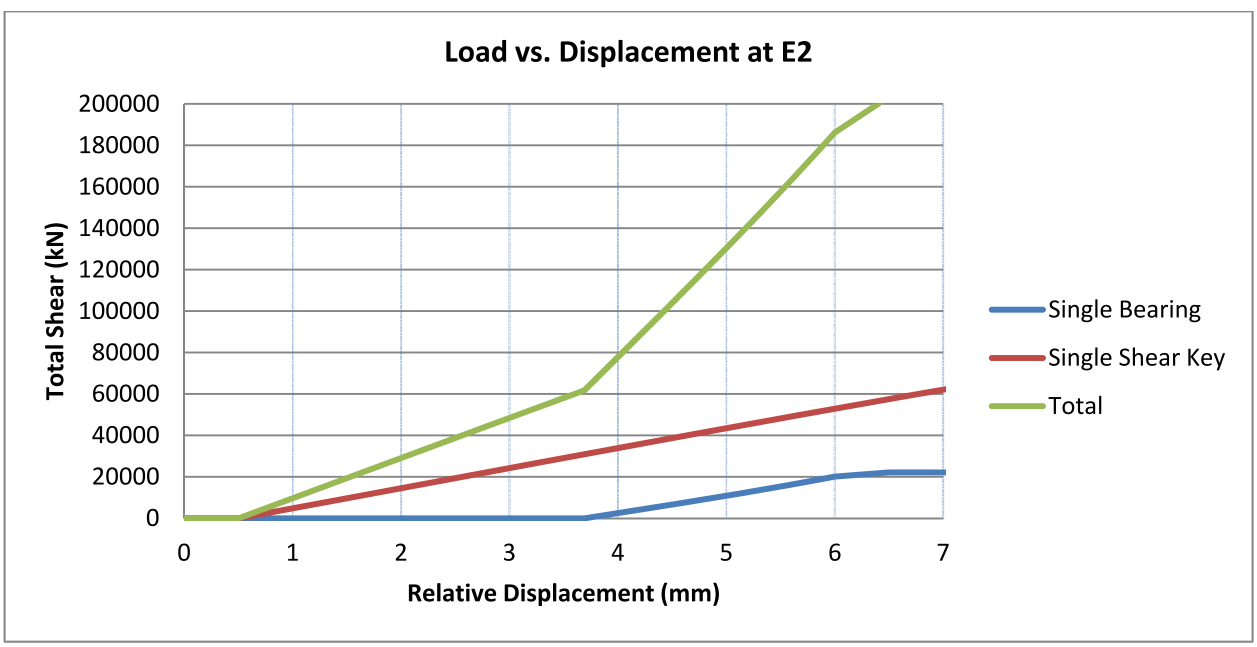


Figure 15 - Combined load vs. displacement for bearings and shear keys

Table 2 - Loads in bearings and shear keys at various displacements

Displ. (mm)	Load (kN)		
	Ea. Bearing	Ea. Shear Key	Total
3.69	0	30883	61767
4.8057	9206	41589	120001
4.8487	9566	42000	122264
5.6799	17059	49882	168001
6.527	22100	57803	204006

Due to the very large stiffness of the force transfer mechanisms at play, very small variations in the size of gaps can have a significant effect on the distribution of load between the components. The previous analyses can be repeated with various assumed gaps in both the shear keys and bearings. Two cases were examined: the gaps in the shear keys and the shimmed bearings are approximately equal and a relatively greater gap in the bearings such that they do not become engaged until approximately 6 mm relative displacement. These are shown in Figure 16. Table 3 and Table 4 list the forces in the components at various displacement levels for these two cases.

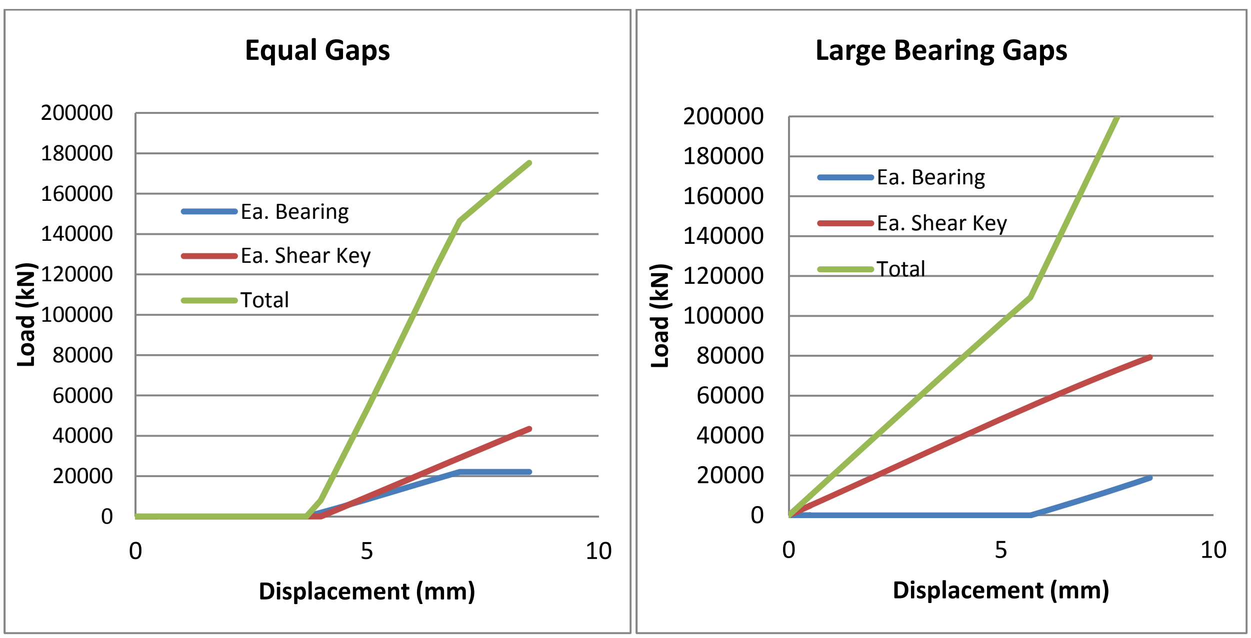


Figure 16 - Load vs. displacement for variable relative gaps

Table 3 - Loads in components with equal gaps in bearings and shear keys

Displ. (mm)	Load (kN)		
	Ea. Bearing	Ea. Shear Key	Total
3.69	0	0	0
4	1998	0	7990
6.4194	18279	23443	120001
6.939	22100	28464	145326
8.1182	22100	39800	168000

Table 4 - Loads in components with 6 mm relative gap between shear key and bearings

Displ. (mm)	Load (MN)		
	Ea. Bearing	Ea. Shear Key	Total
4.3481	0	42000	84000
5.9301	1547	56906	120000
7.0206	8579	66835	168000

Based on the above investigations, the best approach to evaluating the distribution of loads between the shear keys and the bearings is to bound the solution. In addition to the uncertainty in the magnitude of the free play between bearings and shear keys, it is likely that variations exist between the two shear keys and among the four bearings. This would increase the load any one component may experience beyond that found assuming equal distribution among the same components. Another factor is the effect of dynamic impact. During a seismic event, the gaps in these components will close at some non-zero velocity, and this will result in localized impact loading. Simple 2 DOF studies were made to determine the order of magnitude of these impact forces; they were found to be very sensitive to assumed damping levels. Regardless, impact values of 10% or more do not seem impractically large. Therefore, the range of loading to evaluate the shear keys should be from 23.4 MN to 56.9 MN, and for the shimmed bearings from 1.5 MN to 18.3 MN. Based on the evaluation of the capacities of the bearings, there remains approximately 18% reserve capacity before sliding. For the shear keys, although yielding was found to occur at a load in the vicinity of 50 MN, significant capacity exists above that load. Based on the nonlinear finite element analysis discussed in Appendix 2, a maximum capacity of 65 MN appears to be reasonable. This load level limits the amount of inelastic behavior while still taking advantage of the large reserve capacity of the shear keys. Based on this, the reserve capacity of the shear keys is approximately 15% over the SEE demand. Thus, both the bearings and the shear keys have adequate reserve to cover uncertainties in the magnitudes of the loads, including some unequal load sharing within set of like units.

A question might be asked as to why shim the bearings if shear keys S3 and S4 appear to have adequate capacity to carry the full SEE demand by themselves. As discussed above, there are multiple sources of uncertainty regarding the magnitude of the loads and the behavior and robustness of the system is improved by the addition of the transverse shimming.

4.3 Capacity of OBGs and the Pier E2 Cross Girder to Deliver Load to the Shimmed Bearings and Shear Keys, Respectively

4.3.1 Introduction

The steel orthotropic box girder (OBG) and steel cross girder (CG) were evaluated at Pier E2 for the seismic forces applied through the bearings and the S3 and S4 shear keys prior to the completion of the S1 and S2 shear keys. The bearing and shear key locations were examined independently to determine capacities. In some cases, inconsistent conservative assumptions were made regarding loads or load paths. For example, the tension in anchor bolts might be maximized to analyze bearing plates and stiffeners, and then minimized to determine friction from clamping. Another example would be that alternative load paths would be ignored when determining demand in a given component. The demands to determine demand to capacity ratios (D/C) were assumed to be as listed in Table 5. Note that in keeping with the approach to bound possibilities, it has been assumed that either the shear keys or the bearings carry all of the transverse force from the design SEE. For the purpose of evaluating the OBGs and the CG at Pier E2, these loads are local to the individual element and are not additive, i.e. in the temporary shimmed condition the OBGs need only be evaluated for the loads from the bearings and the cross girder need only be evaluated for the loads from the shear keys.

Table 5 - Seismic demands (MN)

	B1	S1	B2	S3	S4	B3	S2	B4
Longitudinal	15	0	15	0	0	15	0	15
Transverse	30	0	30	60	60	30	0	30

4.3.2 Bearings

There are 4 bearings at Pier E2, two under each box girder, as illustrated in Figure 4. The OBG is attached to the bearings using 56 – 2” diameter A354 Grade BD rods anchored into stiffened anchor seats within the OBG. These anchor rods are pretensioned to clamp the bearing onto the thickened bottom flange (key plate) of the OBG. The resulting friction carries the longitudinal and horizontal forces between the OBG and the bearings. The various elements including longitudinal shear plates, transverse webs, and various stiffeners and diaphragms of the OBG at the bearing attachment are illustrated in the design drawings in Appendix 3C.

The stiffened anchor seats were analyzed for the initial tensioning force, as well as the entire local OBG assemblage being analyzed for the transfer of the seismic forces between the OBG and the bearing. The plates are all stiffened compact elements that are assumed to be able to reach their yield point of 345 MPa (50 ksi). Table 6 lists the demand to capacity ratios (D/C) for various elements. The calculations for these values are contained in Appendix 3A. Similar calculations performed by the EOR with comparable results are contained in Appendix 3B.

Table 6 - Demand/Capacity ratios OBG

Component	Force	Demand/Capacity – D/C (%)
1. Key plate	Longitudinal	15%
	Transverse	26%
2. Longitudinal shear plate	Longitudinal	72%
3. Floorbeam webs	Transverse	49%
4. Anchor bolt bearing plate	Bending	66%
5. Center brg stiffener plate	Axial	84%
	Shear	48%
6. Side brg stiffener plate	Axial	35%
	Shear	21%
7. Floorbeam web	Anchor assembly tearout	53%
8. 50mm x 700mm plates	Shear	95%
9. Bearing surface	Friction	95%
10. Local vertical OBG elements	Normal stresses	72%

It should be noted that the bearing surface sliding friction D/C ratio is not for the maximum seismic forces occurring simultaneously in all three orthogonal directions, but is for the maximum uplift of 9.5 MN along with the concurrent transverse and horizontal forces (see calculations in Appendix 3A).

The D/C ratios in Table 6 indicate that OBGs have more than sufficient capacity to resist the demands of the design SEE at pier E2. This conclusion is further supported by the analyses in Sections 4.2.3 and 4.2.4 which indicate that the bearings will not be subjected to the nominal 30 MN under the design SEE.

4.3.3 Shear Keys 3 and 4

Shear keys 3 and 4 are located 2900 mm either side of midspan of the steel cross girder at Pier E2 (see Figure 4 and Design Drawings in Appendix 3C). The CG is attached to the shear keys using 48 – 3" diameter A354 Grade BD rods anchored into stiffened anchor seats and 32 – 3" diameter anchor rods bolting the top plate of the shear key directly to the bottom flange of the CG. These anchor rods are pretensioned to clamp the shear key onto the thickened bottom flange (key plate) of the CG. The resulting friction carries the transverse seismic forces from the CG into the shear key. The various elements of the CG at the shear key attachment including floorbeam webs, diaphragms, and stiffeners are illustrated in the Design Drawings in Appendix 3C.

The stiffened anchor seats were analyzed for the initial tensioning force, as well as the entire local CG assemblage being analyzed for the transfer of the seismic forces between the CG and the shear key. The plates are all stiffened compact elements that are assumed to be able to reach their yield point of 345 MPa (50 ksi). Table 7 lists the demand to capacity ratios (D/C) for various elements. The calculations for these values are contained in Appendix 3A.

Table 7 - Demand/Capacity ratios steel cross girder

Component	Force	Demand/Capacity – D/C (%)
1. Key plate	Transverse	38%
2. Floorbeam webs	Transverse	74%
3. Anchor bolt bearing plates	Interface shear	80%
	Bending	65%
4. Center brg stiffener plate	Axial	81%
	Shear	92%
5. Side brg stiffener plate	Axial	42%
	Shear	37%
6. Diaphragms	Anchor assembly tearout	76%
7. Floorbeam web	Anchor assembly tearout	94%
8. Bearing surface	Friction	60%
9. Local vertical CG elements	Normal stresses	41%
10. Bending+axial global est.	Normal stresses	50%

The D/C ratios in Table 7 indicate that the CG at Pier E2 has more than sufficient capacity to resist the demands of the design SEE. This conclusion is further supported by the analyses in Sections 4.2.3 and 4.2.4 which indicate that the bearings will not be subjected to the 60 MN used in this evaluation under the design SEE.

4.4 Capacity of Strut

4.4.1 Introduction

The effects of the forces corresponding to load path C on the strut at Pier E2 were initially analyzed using the conventional method of strength of materials. A simple static frame analysis was performed to determine the internal forces of the strut. Critical strut sections with forces larger than those reported for load path B were checked according to the AASHTO LRFD Specifications. This assumes (per the scope for this review) that the components were adequately designed by the EOR for this load path. In addition, a lower bound load path wherein all the seismic force is assumed to be transferred through shear keys S3 and S4 was also evaluated.

4.4.2 Structural Analysis

The Lusas frame model shown in Figure 17 was subjected to the loading effects indicated by Table 8 for load path C. The eccentricity between the center of rotation of the bearings and shear keys and the strut centerline was considered in the definition of the model input loads. Table 9 presents the envelope of the sectional forces at the critical bearing and shear key locations including the gravity and axial post-tensioning effects of the strut. The red values indicated in Table 9 correspond to the demand forces that were more critical than their counterparts of load path B. Figure 18 through Figure 20 show the different force effect diagrams of the strut for load path C and the lower bound path.

4.4.3 Sectional Checks

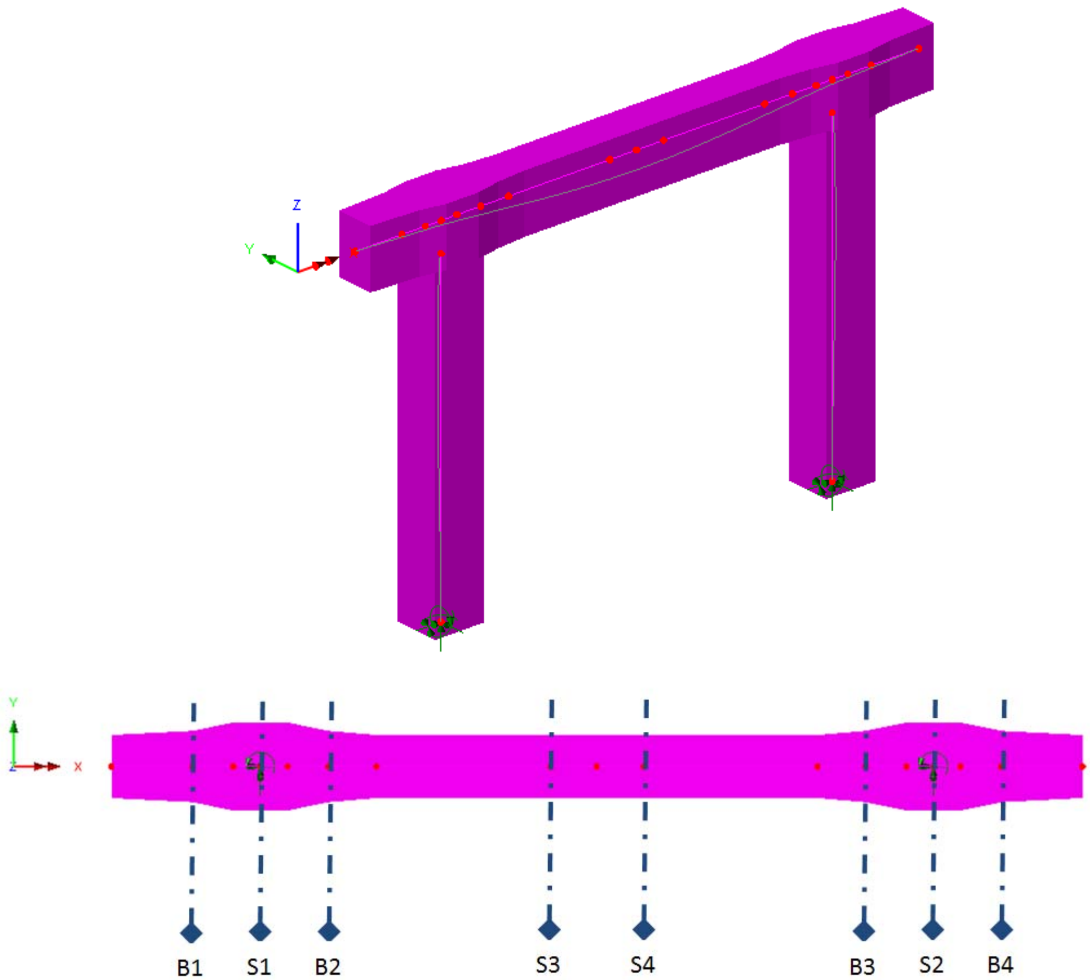
The critical forces from Table 8 and their concurrent effects were compared to the section capacities determined according to the AASHTO LRFD Specifications.

A summary of the principal check findings is presented in Table 9. The red numbers indicate the critical force effect taken from the envelope and the blue shaded numbers mean that the corresponding D/C checks are satisfactory. The complete calculations can be found in Appendix 4.

4.4.4 Results

The results showed that the maximum biaxial flexure interaction equation values were 0.91 and 0.94 at bearing B2 and shear key S2, respectively, while the demand-to-capacity ratios for shear were 0.82 at bearing B3, and 0.58 at shear key S2 (see Figure 17 for location of bearing and shear key sections) indicating more than adequate capacity to resist the SEE forces. The forces on the cantilever portions of the strut resulted in insignificant demands when compared to the available capacity of those components. In fact, the stresses in the cantilevered section of the strut based on this sectional analysis were found to be below the modulus of rupture when the post-tensioning effects were included, meaning the concrete would likely not even crack during an earthquake.

For the lower bound load path, although the minimum axial force in the strut was controlled by this loading case, the biaxial flexure interaction checks showed that this was not a critical condition. Between the columns, the demands are driven by the global frame-action moments and shears such that the change in forces caused by transferring the seismic loads through the bearings had a very minor effect on all but the axial loads, as can be seen in the comparison of the force effect diagrams between load path C and the lower bound path in Figure 18 through Figure 20.



a.

b.

Figure 17 - Overview of Pier E2 Frame model; a. 3D view. b. Identification of Strut Cross Sections

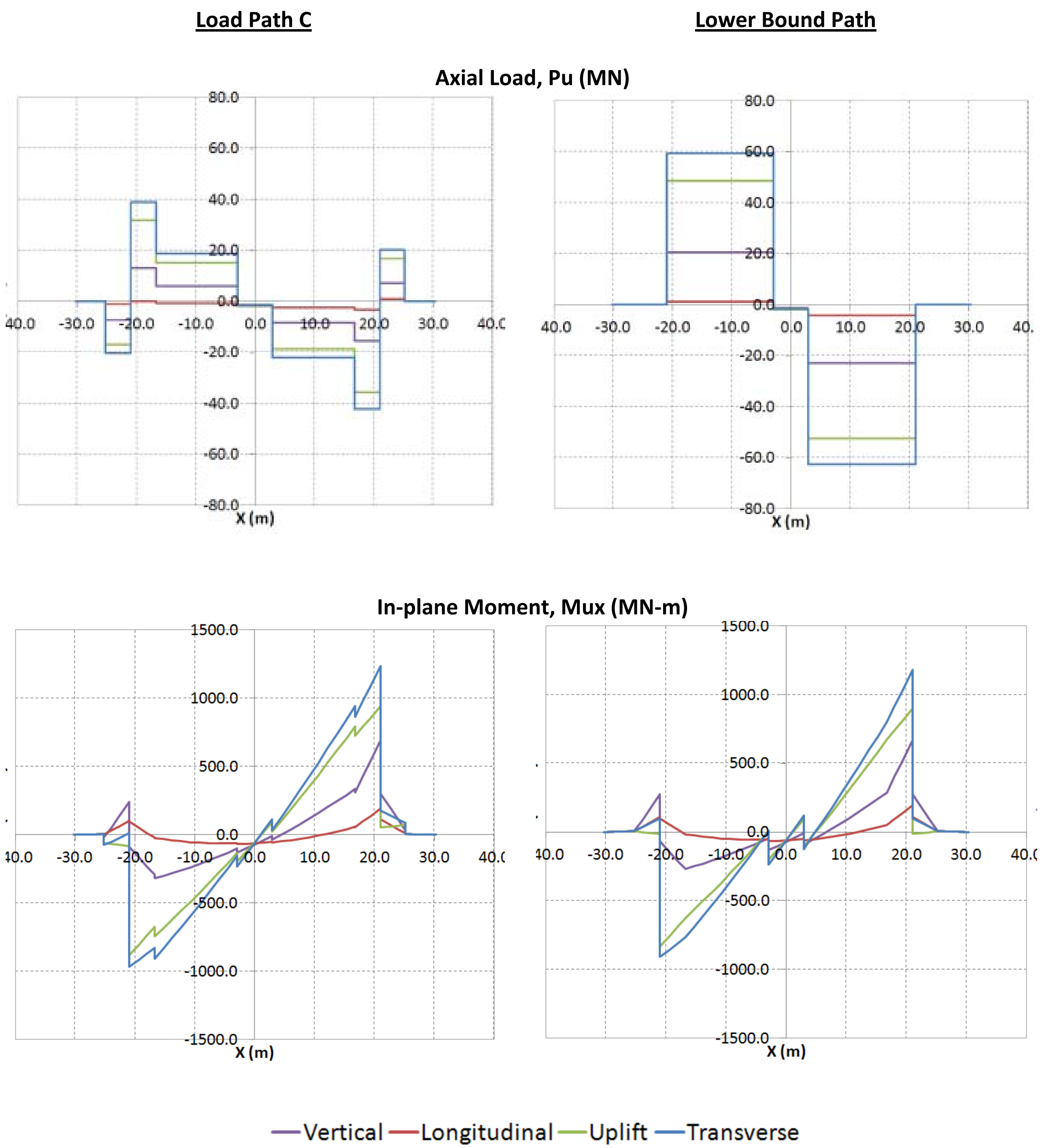


Figure 18 – Force Diagrams for Load Path C and Lower Bound Path (Note: axial post-tensioning effects are not included in the diagrams)

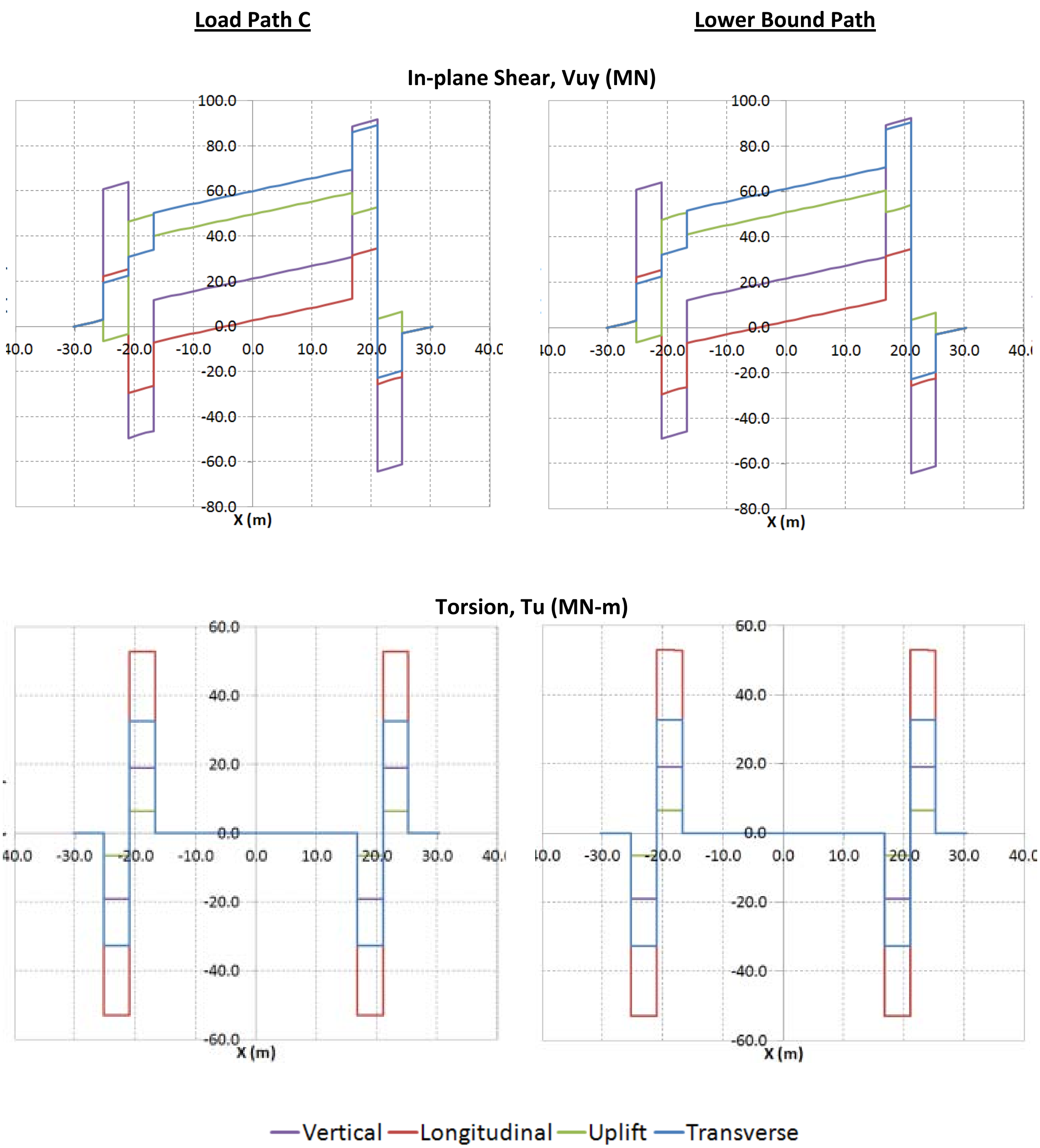


Figure 19 - Force Diagrams for Load Path C and Lower Bound Path

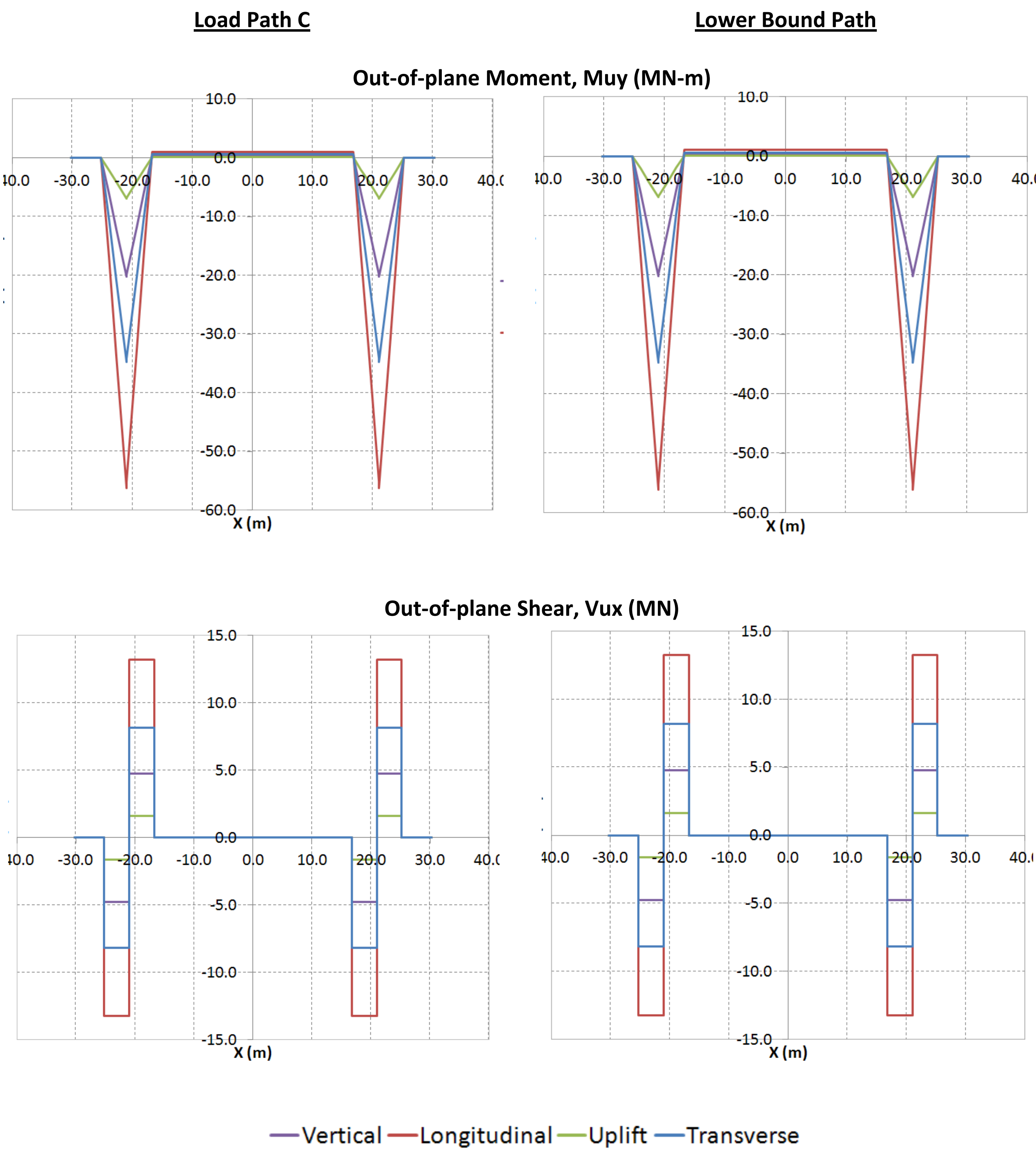


Figure 20 - Force Diagrams for Load Path C and Lower Bound Path

Table 8 - Superstructure loads on Strut at Pier E2

		Bearings B1, B2, B3, B4			Shear Keys S3, S4
		Transverse	Long.	Vertical	Transverse
		KN	KN	KN	KN
Load path B	Transverse	30496	8186	16441	0
	Long.	1340	13232	19255	0
	Vertical	10799	4770	57932	0
	Uplift	25287	1628	-9539	0
Load path C	Transverse	20331	8186	16441	20331
	Long.	893	13232	19255	893
	Vertical	7199	4770	57932	7199
	Uplift	16858	1628	-9539	16858
Lower bound path	Transverse	0	8186	16441	60992
	Long.	0	13232	19255	2680
	Vertical	0	4770	57932	21598
	Uplift	0	1628	-9539	50574

Table 9 - Envelope of sectional forces at bearings and shear keys

			Pu	Vuy	Vux	Tu	Muy	Mux
			Axial	In-plane Shear	Out-of-plane shear	Torsion	Out-of-plane moment	In-plane moment
			MN	MN	MN	MN-m	MN-m	MN-m
Load path B	Bearings B1, B2, B3, B4	max	-114	86	13	57	3	855
		min	-180	-61	-13	-53	-2	-797
	Shear Keys S1, S2	max	-118	89	13	57	-6	844
		min	-180	-52	-13	-49	-58	-1136
Load path C	Bearings B1, B2, B3, B4	max	-104	89	13	53	3	941
		min	-185	-61	-13	-53	-2	-912
	Shear Keys S1, S2	max	-104	92	13	53	-5	1232
		min	-185	-64	-13	-53	-58	-968
Lower bound path	Bearings B1, B2, B3, B4	max	-84	89	13	53	3	799
		min	-206	-61	-13	-53	-2	-770
	Shear Keys S1, S2	max	-84	92	13	53	-5	1177
		min	-206	-64	-13	-53	-58	-913

Table 10 - Critical Sectional Forces and Capacity Checks

Units: Force: MN Moments: MN-m Stress: MPa

		Element:	Bearings				Shear Keys		
		Controlling force:	Mux max	Mux min	Vuy max	Pu min	Mux max	Vuy max	Pu min
		Section:	B3	B2	B3	B3	S2	S2	S2
1.	Sectional loading effects:								
	Axial (if <0 → Tension)	Pu =	165	124	159	206	185	159	206
	(if >0 → Top fiber in tension)	Mux =	941	-912	304	799	1232	688	1177
	Out-of-plane bending moment	Muy =	3	3	2	3	32	18	32
	Torsion	Tu =	0	0	19	0	33	19	33
	In-plane shear	Vuy =	70	50	89	71	89	92	90
	Out-of-plane shear	Vux =	0	0	5	0	8	5	8
2.	Normal stresses assuming elastic behavior:								
	(if >0 → compressive stress)	fc 1 =	42	-29	17	38	44	26	43
		% f'c =	75%	53%	32%	68%	80%	47%	78%
		fc 2 =	-29	39	-6	-22	-33	-17	-30
		% f'c =	53%	70%	10%	40%	59%	30%	55%
3.	Axial and flexural force effects:								
		Pnx =	172	174	209	248	195	177	227
		Mnx =	1239	1274	1291	1341	1528	1497	1577
		ey ratio =	1.00	1.00	1.00	1.00	1.00	1.00	1.00
		Pny =	1063	1209	1065	1082	997	845	1112
		Mny =	31	27	26	11	712	873	535
		ex ratio =	1.00	1.00	1.00	1.00	1.00	1.00	1.00
		IE =	0.86	0.91	0.27	0.72	0.94	0.53	0.89
4.	Shear and torsional force effects:								
	Torsional effects:	φTn=	82	82	94	88	46	52	47
		Tu/φTn =	0.00	0.00	0.20	0.00	0.71	0.37	0.69
	In-plane shear:	φVn =	88	87	109	97	135	158	139
		Vu/φVn =	0.79	0.58	0.82	0.73	0.66	0.58	0.65
	Out-of-plane shear:	φVc =	51	51	51	51	63	63	63
		Vu/φVc =	0.00	0.00	0.08	0.00	0.12	0.07	0.12

4.5 Transfer of Horizontal Load From Bearing Base Plate to Strut to Column

Due to the size and localized loading conditions of the concrete pier strut, strut-and-tie models were developed for the two regions shown in Figure 21 as an additional check beyond the sectional interaction values of Section 4.4. Since the controlling loading case corresponds to the transverse forces, only two dimensional models subjected to these forces were considered in this study.

To determine an appropriate strut and tie model that accurately represents the flow of forces in these disturbed regions, an elastic analysis of a plane stress model of the strut was performed in Lusas to obtain the principal stresses in the uncracked regions. Strut and tie configurations were then developed based on the flow of forces in these models (see Figure 22). In order to bound the potential loads applied at the bearings and shear keys, the bearings were evaluated for the forces from load path B, while shear keys 3 and 4 were loaded with the lower bound forces, assuming all transverse seismic force is transmitted through these two shear keys. Results of the frame analysis were used to determine the boundary loads applied to the strut and tie models. Results of the analysis indicate that there is sufficient capacity to accommodate the changes in load path caused by the shimming of the bearings. One anomalous result was encountered regarding the tie capacity in the tension side of the pier column.

Since this is outside of the scope of this review, it was not pursued but was reported to the EOR for further review.

The complete calculations corresponding to the strut-and-tie models are presented in Appendix 5.

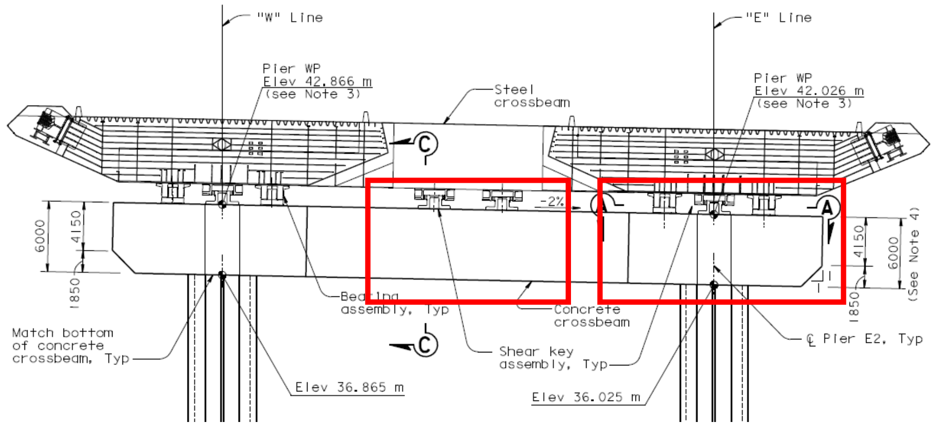


Figure 21 - Strut regions analyzed with strut-and-tie models

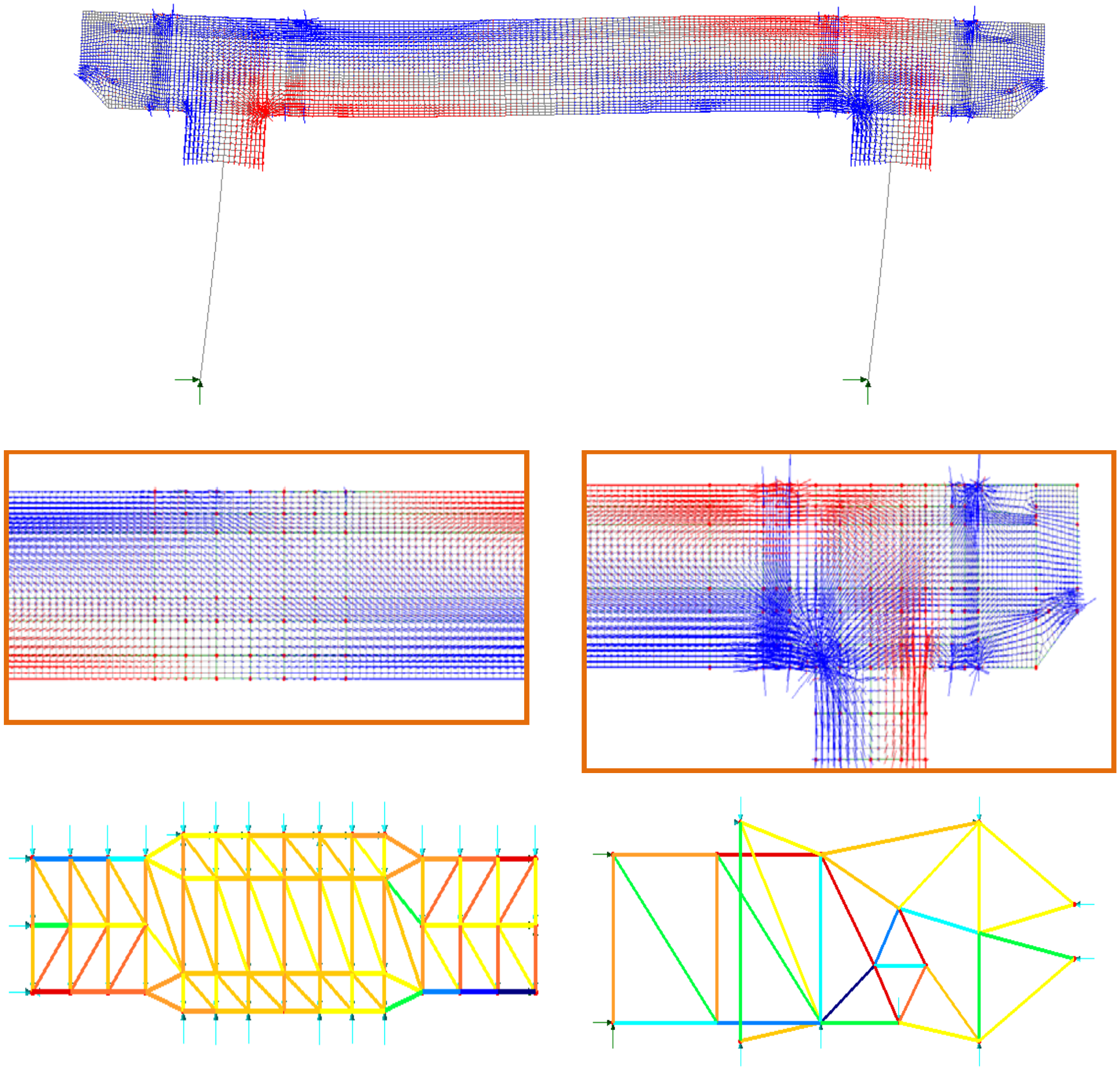


Figure 22 - Principal stress trajectories and strut-and-tie models (blue: compression, red: tension)

4.6 Potential to Damage the Permanent Bearings or Other Components if Shims are Installed

The analysis of Pier E2 in Section 4.4 shows that the service and seismic design forces in the pier are not changed significantly by the redirection of seismic forces resulting from the shimming. Therefore the response of the concrete strut and columns are not expected to be different with or without the shims. Based on this subjective assessment of the global effects, combined with the quantitative assessment of the local elements in the load transfer zone, it is concluded that there does not appear to be any reasonable possibility of damaging the bridge as a result of installing the shims, other than the possibility of damaging the paint system either when the shims are inserted or as the bridge rotates in service

5. Conclusions and Recommendations

The exact distribution of forces in the bearings and shear keys, and hence the demands on other parts of the load transfer zone at Pier E2 which is the scope of this investigation, is highly dependent on the planned and accidental tolerances (gaps) between various components. A precise quantification of the forces is, for practical purposes, unknowable. It is, however, possible to make estimates of the reasonable range of possibilities rather than placing reliance on a single solution; MM's assessment is based on that approach. Various assumptions have been used to place reliance on the set of shear keys and on the set of shimmed bearings, separately and in combination. Analysis of the resulting distribution of forces described herein formed the basis for the conclusion and recommendation below.

Modjeski and Masters, Inc. (MM) concludes that the concept of temporarily shimming the bearings at Pier E2 of the San Francisco-Oakland Bay Bridge as described in the July 15th information package entitled "Seismic Evaluation of SAS at E2 Bent Prior to Completion of Shear Keys S1 and S2", and the proposed details, will provide more than sufficient capacity between the superstructure and the strut at Pier E2, including the strut itself, to resist the design Safety Evaluation Earthquake. The service and seismic design forces in Pier E2 are not changed significantly by the redirection of seismic forces resulting from the shimming. Therefore, the response of the concrete strut and columns are not expected to be different. Other than the possibility of scratching the paint system either when the shims are inserted or as the bridge rotates in service, there does not appear to be any reasonable possibility of damaging the bridge as a result of installing the shims. Assuming that the rest of the structure has been properly designed, we conclude that the safety of the traveling public is improved by moving traffic on to the new bridge and we recommend that action.